SANTA CLARA COUNTY FLOOD CONTR
AND WATER CONSERVATION DISTRIC

GUADALUPE RIVER
Reach 1-2 & Gold St Br
SOIL

SOIL INVESTIGATION FOR THE PROPOSED GUADALUPE RIVER PROJECT - REACHES 1 and 2

Santa Clara County, California

WOODWARD, CLYDE, SHERARD AND ASSOCIATES
Consulting Soil and Foundation Engineers

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REPORT

to

MR. GEORGE PRIME, SUPERVISING ENGINEER
SANTA CLARA COUNTY FLOOD CONTROL
AND WATER CONSERVATION DISTRICT
County Office Building
San Jose, California

on

SOIL INVESTIGATION FOR THE PROPOSED GUADALUPE RIVER PROJECT - REACHES 1 and 2

Santa Clara County, California

by

WOODWARD-CLYDE-SHERARD AND ASSOCIATES
Consulting Soil & Foundation Engineers
Oakland, California

WOODWARD - CLYDE - SHERARD & ASSOCIATES

P.O. BOX 8253 • OAKLAND 8, CALIFORNIA • TELEPHONE HIGATE 4-1256

Consulting Soil and Foundation Engineers

OAKLAND, CALIF.
DENVER, COLO.
OMAHA, NEB.
KANSAS CITY, MO.
PHILADELPHIA, PA.
MONTCLAIR, N. J.
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SAN DIEGO, CALIF.
NEW YORK, N. Y.

December 30, 1960

Job No. 4246

Mr. George Prime, Supervising Engineer
Santa Clara County Flood Control
and Water Conservation District
County Office Building
San Jose, California

Dear Sir:

In accordance with the request of Mr. P. Creegan, we are submitting our final report of a soil investigation for the Guadalupe River Project, Reaches 1 and 2 in Santa Clara County, California.

The enclosed report supplements the preliminary report submitted on November 18, 1960 which contained information and recommendations that were subject to change. A report regarding the Southern Pacific railroad bridge relocation is submitted under separate cover.

The Staff Engineers assigned to this project by our firm were Mr. George E. Hervert and Mr. Don Westphal.

Very truly yours,

WOODWARD-CLYDE-SHERARD & ASSOCIATES

By Stanley F Gizienski

Stanley F. Gizienski, R.E. 10352

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4 cc - Creegan and D'Angelo

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REPORT

to

MR. GEORGE PRIME, SUPERVISING ENGINEER
SANTA CLARA COUNTY FLOOD CONTROL
AND WATER CONSERVATION DISTRICT

on

SOIL INVESTIGATION FOR THE PROPOSED GUADALUPE RIVER PROJECT - REACHES 1 and 2

Santa Clara County, California

SCOPE

This supplements our previous report dated November 18, 1960 entitled "Preliminary Report on Soil Investigation for the Guadalupe River Project - Reaches 1 and 2". Included in this report are the final results of laboratory tests and further recommendations which confirm and/or supplement those given in the preliminary report. Logs and locations of test borings, and profiles are contained in the initial report. In this report we are including logs of borings for the new railroad bridge alignment since they supply supplemental information in the dump area near Alviso.

SOIL TESTS

GENERAL

Undisturbed and disturbed samples of soil taken from the borings were visually inspected and representative materials selected for testing. Tests were made on these selected materials to evaluate the characteristics pertaining to strength, consolidation, and permeability in order to establish the criteria that will be used in design.

LABORATORY TEST PROCEDURES

Classification tests were made on undisturbed and disturbed samples in accordance with the ASTM test procedures designated below. The soils were classified by the Unified Soil Classification System.

Laboratory compacted samples were tested at 95 percent of the maximum density and at the optimum water content as determined by the ASTM procedures indicated.

The sack samples of the disturbed material were first visually classified in the laboratory and separated into groups of similar material. Samples from each group were then designated for testing by sieve analysis or Atterberg Limits, or both, and classified. After classificiation, similar materials were combined into composite samples for further testing.

Unconfined Compressive Strength. Selected undisturbed samples were tested for unconfined compressive strength. The samples were extruded from the sample tubes, trimmed, measured, and weighed before subjecting the samples to compressive loads while unconfined. The dry densities and moisture contents were also determined for each of these samples

A summary of the results of these tests was given in Table 1 of the previous report and were also shown on the logs of borings.

Sieve Analyses and Hydrometer Analyses. Representative samples of similar materials taken from the auger holes were screened through sieves of various sizes in accordance with procedures outlined in ASTM Test Designation D 422-54T. All fine material from the samples was first washed over a No. 200 mesh screen in order to determine the amounts finer than the No. 200 mesh size. Hydrometer analyses were made on material passing the No. 10 mesh sieve.

Results of mechanical analysis tests on undisturbed samples are shown on Figs. 45 through 47; results on disturbed material are shown on Figs. 48 through 50.

Atterberg Limits. The Atterberg Limits of the finer size material were determined in accordance with ASTM Designation D 423-54T and D 424-54T on material passing the No. 40 mesh sieve. Classification of the material according to plasticity characteristics are shown on Figs. 51 and 52.

Moisture Content. Moisture content determinations were made on the samples by weighing the material at the compacted or natural moisture content, drying in an oven at a temperature of 110° C, and then weighing when completely dried.

Specific Gravity. Specific gravity tests were made on the samples tested for compaction. The test procedure was in accordance with ASTM Designation D 854-52.

Compaction. Compaction tests were made in accordance with the procedures outlined in "Moisture-Density Relations of Soils" laboratory test, ASTM Designation D 698-57T (Method A). The tests were performed on material passing the No. 4 sieve (-#4).

The results of these tests are shown on Figs. 53 and 54.

Permeability Tests. Falling head permeability tests were made on undisturbed samples. Undisturbed samples were 2 in. in diameter. The results are summarized in Table 1.

Triaxial Shear Tests. Unconsolidated-undrained triaxial shear and consolidated-undrained triaxial shear tests with pore pressure measurements were made on specimens of recompacted soil and on undisturbed material. All specimens were trimmed to a nominal 2- or 2.8-in. diameter and to a 4- or 6-in. height. The trimmed specimens, protected by membranes, were placed in a triaxial cell and tested under various confining pressures.

In the case of a consolidation phase the specimens were saturated with de-aired distilled water under the confining pressure. After consolidation and saturation drainage was completely shut off by closing the ports and an initial pressure was introduced into the specimens to check the functioning of the pore pressure device. The specimens were then sheared at a constand rate of strain of 0.005 in/min. As the test progressed vertical deformations and internal pore water pressures were measured. The loadings were continued until failure of the specimens.

The unconsolidated-undrained shear test specimens were prepared as described above. The specimens were placed in the triaxial apparatus, the ports at the top and bottom of the specimens were closed and a confining pressure applied. The specimens were sheared at a constant rate of strain of 0.060 in./min until failure occurred.

The results of consolidated undrained triaxial tests on compacted material are shown on Figs. 55 and 56 and are summarized in Table 2; the results on undisturbed samples from Reach 2 are shown on Figs. 57 through 59 and are summarized on Table 3. Results of consolidated-undrained shear tests on undisturbed samples in Reach 1 are shown on

Figs. 60 through 65 and are summarized on Table 4. The unconsolidated undrained test results are summarized in Table 5.

Consolidation. Undisturbed samples were tested in the consolidometer. All specimens were trimmed into consolidometer rings 1.9 or 2.8 in. in diameter and 1 in. high. Vertical loads were applied in increments and the rate of vertical deformation measured for each increment. The tests were performed in the submerged state from the start of loadings. The test results are shown on Figs. 66 and 67 for Reach 2 and Figs. 68 through 76 for Reach 1.

REACH 2

(Holes 1 through 17)

GENERAL

The general soils conditions along with the logs of borings are described in the previous report. General design criteria and problems that may be encountered in construction of the channel in this reach are discussed below.

SHEAR STRENGTH

Based on a review of the laboratory test results and the logs of borings, showing the foundation conditions along this reach, the following shear strengths are recommended for use in designing and analyzing slope problems:

TYPE MATERIAL	COHESION, psf Ce	ANGLE OF INTERNAL FRICTION, Ø, DEGREES
Compacted Fill	300	26.5 φe
Foundation Soils	200	19.3
Dumped or Uncompacted Fill	0	15

The values quoted above represent an average shear strength, that in our opinion, can be encountered in many portions of Reach 2. The area investigated is made up of alluvial deposits and many of the soils encountered have not been fully consolidated, therefore, strength values can vary considerably. As shown by the profiles of the logs of borings layers are decontinuous and peat zones and high water table conditions can be expected to be encountered. The values quoted above for uncompacted fill is not based on laboratory tests and any tests made would show a wide and misleading set of values.

SHRINKAGE OF VOLUMES

Based on the laboratory results and considering the quality of compacted fill to be placed, it is recommended that a shrinkage factor of at least 20 percent be used in estimating quantities for the soils in Reach 2. This figure includes settlement during construction, shrinkage and soil loss.

SETTLEMENT

Evaluation of the soil conditions as shown on the logs of borings and results of laboratory consolidation tests indicate that some settlement beneath new levees should be expected; however, it is estimated that 30 to 50 percent of the ultimate settlement will occur during construction. The soils in Reach 2 contain a considerable amount of sand and silt and should consolidate during placement of the levee fill. It is estimated that about 2 in. of settlement will occur during construction and about 3 in. more over a long period of time. The magnitude of these is small and should have no effect on the operation of the levees in Reach 2.

SLOPE RECOMMENDATIONS

The following slope inclinations are recommended for the area between Montague Road and the State Highway bridge except between Stations 195 and 206 as discussed below.

TYPE OF SLOPE	SLOPE INCLINATION (Horizontal to Vertical)
Native Cut Slopes	2 to 1
Compacted Levee Slopes	2½ to 1
Compacted Fill Slope in New Channel	2½ to 1

In establishing the slope inclination consideration must be given not only to the stability under draw down conditions but also to ease of maintenance and control or erosion by flowing water.

Our borings indicated that soft and weak soils may be encountered in the interval of Station 195 to 206 (original stationing). We recommend that consideration be given to using flatter cut slopes in this area as recommended for Reach 1.

SEEPAGE

Our investigation indicates the presence of silty sands in Holes 4, 8, and 17. Laboratory permeability tests show that the coefficient of permeability of these soils varies from 1.3×10^{-4} to 2.2×10^{-6} cm/sec. This range of permeability indicates poor drainage characteristics. It is our opinion that these soils are poor draining and being located at a considerable depth (12 ft) below the ground surface we do not anticipate difficulty with seepage. It is possible that, during construction, granular layers may be encountered at higher elevations and it is recommended that a 5 ft thick compacted layer of impervious soil be placed over the pervious strata on the channel side of the levee.

CONSTRUCTION

In order to provide a firm base for the placement of fill all organic materials should be removed in the stripping operation. Where the old channel will become a part of the new channel and fill is to be placed it is recommended that the soft deposits be removed; borings indicate that this type of excavation may vary from 5 to 15 ft in depth.

Care should be taken during construction to avoid excessively steep temporary construction slopes or undercutting of existing or finished slopes. Some local sloughing will probably occur when boggy areas are encountered.

RIPRAP

The new aligment of the channel must of necessity include some bends. It is recommended that riprap be placed in these reaches to prevent erosion and undercutting of the new slopes.

REACH 1

(Holes 18 through 44)

GENERAL

This reach contains weak foundation soils locally termed "Bay Mud". The logs of borings and a description of the soils are contained in our previous report. Additional borings made for the railroad relocation show that the debris and garbage fill in the dump at the intersection of the new railroad and new channel is 6 to 10 ft thick and is underlain by about 15 to 20 ft of soft Bay Mud. Logs of these borings are shown in Figs. 38 through 44. The problems which will be encountered in constructing levees on these foundations and of the Bay Mud are quite different from those expected in Reach 2.

SHEAR STRENGTH

A review of the triaxial shear and unconfined compression tests show that the Bay Mud has a wide range of strengths which are primarily dependent on the internal pore pressures and degree of consolidation under superimposed loads. Evidence in the field indicates that some of the slopes in the Bay Mud are quite steep both above and below the water line. Under construction conditions, however, the soils become quite weak due to development of pore pressures and remolding of the soils.

It is recommended that the following shear strengths be adopted for use in analyzing slopes in this reach.

Material	Cohesion, psf	Tan Ø
Bay Mud	200	11.3

SHRINKAGE

It is assumed that most of the construction within Reach 1 will be done by placing uncompacted fill by water borne equipment. It is recommended that a shrinkage factor of 30 percent be used for estimating earth quantities in levee construction. This figure includes settlement, shrinkage and soil loss.

SETTLEMENT

Since the alignment of the channel has not been definitely established at this time and the conditions influencing settlement are quite variable, Figs. 77 and 78 have been prepared for general use. The data on Fig. 77 may be used for determining the ultimate settlement if the thickness of the compressible layer and surcharge are known. The data on Fig. 78 give time-settlement relationships for a given thickness of compressible material and a desired degree of consolidation. As an example, a 15 ft layer of compressible material surcharged by a load of 600 psf would result in an ultimate settlement of 15.6 in. (fig. 77). From Fig. 78 the time required for 50 percent consolidation of the 15 ft layer is 80 months. With the aid of these curves it should be possible to estimate the settlement at a desired point in the levee system. It should be noted that increasing the surcharge results in larger ultimate settlements and will also tend to make slopes less stable. It is our opinion that continual maintenance will be required along the levee system to maintain the top of the levee at a desired elevation. One factor that has not been taken into consideration in the settlement curve is the presence of peat. Our borings encountered peat beneath the existing levees particularly along the left bank and at considerable depths. It is estimated that the ultimate settlement will be increased about 50 percent in peaty areas. In general, when peat can be seen during construction, it should be removed to a practical depth to reduce the settlement. The deeper layers of peat cannot be removed economically and if they are located below the water table consolidation and deterioration of the peat will be somewhat retarded.

SLOPE RECOMMENDATIONS

Shear strength tests on the soft (Bay Mud) soils in this reach show that they are very weak. It is recommended that channel cut slopes not exceed an inclination of 5 to 1 in the areas of Bay Mud. Fill slopes under 10 ft in height should not exceed an inclination of 4 to 1 and those over 10 ft should not exceed 5 to 1. There are intervals in this reach where the soft clays are not encountered. In such cases steeper channel cut slopes may be used and if the levees are constructed by land equipment and compacted they may also be constructed with steeper slopes. The areas in question are the north banks of the new channel between Stations 220+00 and 239+00 and possibly the south bank between Stations 230 and 239+00. In these intervals slope recommendations as given for Reach 2 may be followed.

The logs of borings 23 and 25 indicate that the soft clay deposits between Stations 293 and 320 are shallow. Consideration should be given to excavating the Bay Mud and supporting the new levees on the underlying stiff clay.

SEEPAGE

Our borings in this reach indicate that granular layers are encountered well below the present channel bottom and the possibility of developing piping or sand boils appears to be very remote. In the dump area, however, the fill is hetergeneous, contains voids, and may develop erratic seepage channels. It is, therefore, recommended that an impervious zone or blanket, at least 10 ft thick be placed on the channel side slopes where the dump is encountered. This procedure will require extensive stripping of the dump area adjacent to the channel.

CONSTRUCTION

Where new levees overlap existing levees it is recommended that at least 5 ft of the existing material be stripped before placing the new material. Many of the existing levees contain peat which will magnify the settlement in the future.

It is anticipated that some slides will develop during construction. It is recommended that where extensive fills are placed that the area be first filled to about 2 ft above the water line and allowed to settle and gain some strength for about one month. Construction of the levee toe should be started at least 30 ft back from the top of the excavated slope and the material placed in 2 ft lifts on as steep slopes as possible to the high tide level and the material allowed to set for an additional month. After this period the berm area and levee may be completed to the final height and inclination by placing the material in 2 ft lifts at intervals of 2 weeks.

GOLD STREET BRIDGE

GENERAL

The logs of borings and generalized profile of the conditions along the Gold Street Bridge alignment are contained in the previous report. Soil conditions were also described in the preliminary report. Several schemes for reducing the settlement at the abutments and estimated pile length are presented below.

SETTLEMENT

According to the present plans about 18 ft of road fill will be placed adjacent to the channel banks for the highway approach fills. Since the highway will pass through the dump area, settlement of the approaches will be influenced by the thickness of the dumped trash and garbage material as well as the Bay Mud. Estimates of ultimate settlement for various thicknesses of Bay Mud are shown on Fig. 79 and the time required for various degrees of consolidation is shown on Fig. 80. No consideration has been given to the dump material since it will have to be removed prior to placing the fill.

TREATMENT OF APPROACHES

Surcharge Method - Where 17 ft (2100 psf) of fill is placed over 20 ft of compressible Bay Mud approximately

17.5 in. of ultimate settlement is anticipated. If an additional surcharge of 5 ft of fill or a total thickness of 22 ft (2700 psf) is placed an ultimate settlement of 21 in. is anticipated. Leaving the additional surcharge in place for 1 yr permits 6 in. of settlement to occur. Removal of the additional surcharge after a period of one year would then reduce the settlement of the approach fill after construction to 11.5 in.

In our opinion surcharging an area can be beneficial and can be accomplished economically if sufficient time is allowed to consolidate the underlying soils. In the present case it appears that a surcharge would have to remain in place for 5 yrs to reduce the settlement of the approach fill to 6 in. or less.

Blasting Method - We have considered the possibility of blasting the Bay Mud after the approach fills have been placed. After the fills are in place explosive charges beneath the fill would be set off and the Bay Mud displaced laterally thus permitting the approach fill to sink into the resulting void. Experience by others in this type of operation indicates that the blasting does not create uniform conditions and erratic settlement continues. In our opinion this method is the least desirable.

Sand Drains Method - Studies indicate that if the area beneath the approach fills were perforated with vertical sanddrains extending through the Bay Mud to the underlying sand and gravel, settlement could be accelerated. The drains would have to be at least 12 in. in diameter, spaced 15 ft on centers and backfilled with free draining sand. A gravel or sand blanket about 3 ft thick would then have to be placed over the surface to connect the drains and ensure positive drainage. The approach fills would then be placed over the sand on gravel blankets and settlement allowed to continue. Under the above scheme a compressible layer would consolidate 50 percent of the ultimate amount in 6 months and 75 percent of the ultimate in one year.

Overexcavation Method - Another scheme for reducing the settlement to tolerable amounts involves excavating the Bay Mud beneath the approach fills and backfilling the excavation with a granular type of material below the water level and compacted fill above. This procedure would remove the compressible material and most of the settlement that will take place will occur during placement of the approach fills. In order to prevent seepage, the channel slope would have to be sealed with an impervious blanket.

SIDE SLOPES

In all the schemes for reducing settlement outlined above except the overexcavation, side slopes of the approach fills should not exceed an inclination of 5 to 1 in areas where Bay Mud is indicated. In the case where the Bay Mud is replaced with a stronger foundation material, the side slope should not exceed an inclination of 2½ to 1.

PILES

The pile type of foundation was the only one considered for this project. For your convenience in estimating pile lengths Figs. 81, 82, and 83 have been prepared for 9½-in. step-taper cast-in-place piles, 10¾-in. pipe piles and 12-in. octagonal prestressed concrete piles. Since the depth to the supporting strata varies the figures have been drawn to show the pile lengths at the north and south abutment and in the center of the proposed channel. Settlement of the approach fills will place additional load on the piles. We have included 12 tons of drag in the curves shown in Figs. 81, 82, and 83. The ultimate carrying capacity takes credit both for end bearing and friction. It is recommended that a safety factor of at least 2 be incorporated into the design of the piles. The use of batter piles will be required at the approaches and no studies are planned until the method of treatment beneath the approach fills is selected.

LIMITATIONS

The recommendations made in this report are based on results of test borings spaced at considerable distances. It is anticipated that some variations in soil conditions will be revealed between borings during the actual excavation of the channel. It is our opinion that the exploration program made for this investigation provides a basis for proceeding with design, however, the design should permit some modifications of slopes in the field. Close liaison during construction should be maintained between the Designing Engineer, the Soils Engineer and the Contractor.

TABLE 1

SUMMARY OF PERMEABILITY TESTS ON UNDISTURBED SAMPLES

SAMPLE NUMBER	SAMPLE ELEVATION, FT.	TYPE OF MATERIAL	COEFFICIENT OF PERMEABILITY (cm/sec)
4-3	8	Silty Sand	1.3×10^{-4}
8-3	1	Silty Sand	4.7×10^{-5}
17-3	-12	Silty Sand	2.2×10^{-6}

TABLE 2 - SUMMARY OF TRIAXIAL SHEAR TESTS ON RECOMPACTED MATERIAL,

CONSOLIDATED UNDRAINED - REACH 2

				TOTAL STRESS		EFFECTIVE STRESS	
SAMPLE NUMBER	TYPE OF MATERIAL	INITIAL WATER CONTENT,%	INITIAL DRY DENSITY, PCF	COHESION PSF	ANGLE OF INTERNAL FRICTION,Ø°	COHESION PSF	Ø°
A-1	Clayey Sandy Silt, ML-CL	17.1	106.8				
A-2		17.0	106.8	400	21	0	34
A-3		16.9	106.8				
B-1	Silty Clay, CH	20.2	100.0				
B-2		19.5	100.5	200	20	0	34.5
B-3		20.2	100.0				

TABLE 3 - SUMMARY OF TRIAXIAL SHEAR TESTS ON UNDISTURBED SAMPLES,

CONSOLIDATED UNDRAINED - REACH 2

				TOTAL STRESS		EFFECTIVE STRESS		
SAMPLE NUMBER	TYPE OF MATERIAL	INITIAL WATER CONTENT,%	INITIAL DRY DENSITY, PCF	COHESION PSF	ANGLE OF INTERNAL FRICTION,Ø°	COHESION PSF	Ø°	SAMPLE ELEVATION FT.
7-2-2	Sandy Clay	33	90					
7-2-3		33	89	200	26	0	35	-2
7-2-1		33	89					
11-3-3	Sandy Clay	26	100					
11-3-4		23	104	0	21	0	33	-5
11-3-2		33	90					
16-2-1B	Bay Mud	90	50					
1602-1A		90	50	0	14	0	26.5	-7

TABLE 4 - SUMMARY OF TRIAXIAL SHEAR TESTS ON UNDISTURBED SAMPLES, CONSOLIDATED UNDRAINED - REACH 1

SAMPLE NUMBER	TYPE OF MATERIAL	INITIAL WATER CONTENT, %	INITIAL DRY DENSITY, PCF	TOTAL STRESS COHESION PSF	TOTAL STRESS ANGLE OF INTERNAL FRICTION, Ø°	EFFECTIVE STRESS COHESION PSF	EFFECTIVE STRESS Ø°	SAMPLE ELEVATION FT
19-2-3	Bay Mud w/Peat	86	51	0	13	0	28	-10
19-2-2		94	47					
19-2-4		77	55					
22-1-3	Sandy Clay	65	61	100	19	0	35	-6
22-1-2		45	71					
31-3-4	Bay Mud w/Shells	34	87	0	18	0	32	-7
31-3-2		66	57					
31-3-3		37	83					
35-3-3	Bay Mud w/Peat	51	70	0	17	0	37	-8
35-3-2		56	67					
35-3-1		88	49					
38-3-4	Bay Mud	34	88	0	19	0	36	-9
38-3-2		43	79					
38-3-3		38	83					
B3-2-2	Bay Mud	26	97	0	20	0	34	-17
B3-2-4		30	95					
B3-2-3		25	101					

TABLE 5 - SUMMARY OF TRIAXIAL SHEAR TESTS ON UNDISTURBED SAMPLES,

UNCONSOLIDATED UNDRAINED - REACH 1

SAMPLE NUMBER	TYPE OF MATERIAL	CONFINING PRESSURE TSF	SHEAR STRENGTH PSF	INITIAL DRY DENSITY, PCF	INITIAL WATER CONTENT,%	SAMPLE ELEVATION, FT
18-1a	Bay Mud w/Peat	5.0	178	45	105	-7
18-1b	Bay Mud w/Peat	10.5	192	49	93	-7
26-1	Silty Sand	10.6	236	86	39	-8
29-4	Bay Mud w/Peat	12.4	192	50	90	-5
34-1	Silty Sand	3.6	4460	94	33	-11
R2-2	Bay Mud w/Peat	4.0	386	77	44	-2
R2-3a	Bay Mud w/Peat	2.0	408	91	31	-7
R2-3b	Bay Mud w/Peat	0	320	91	32	-7

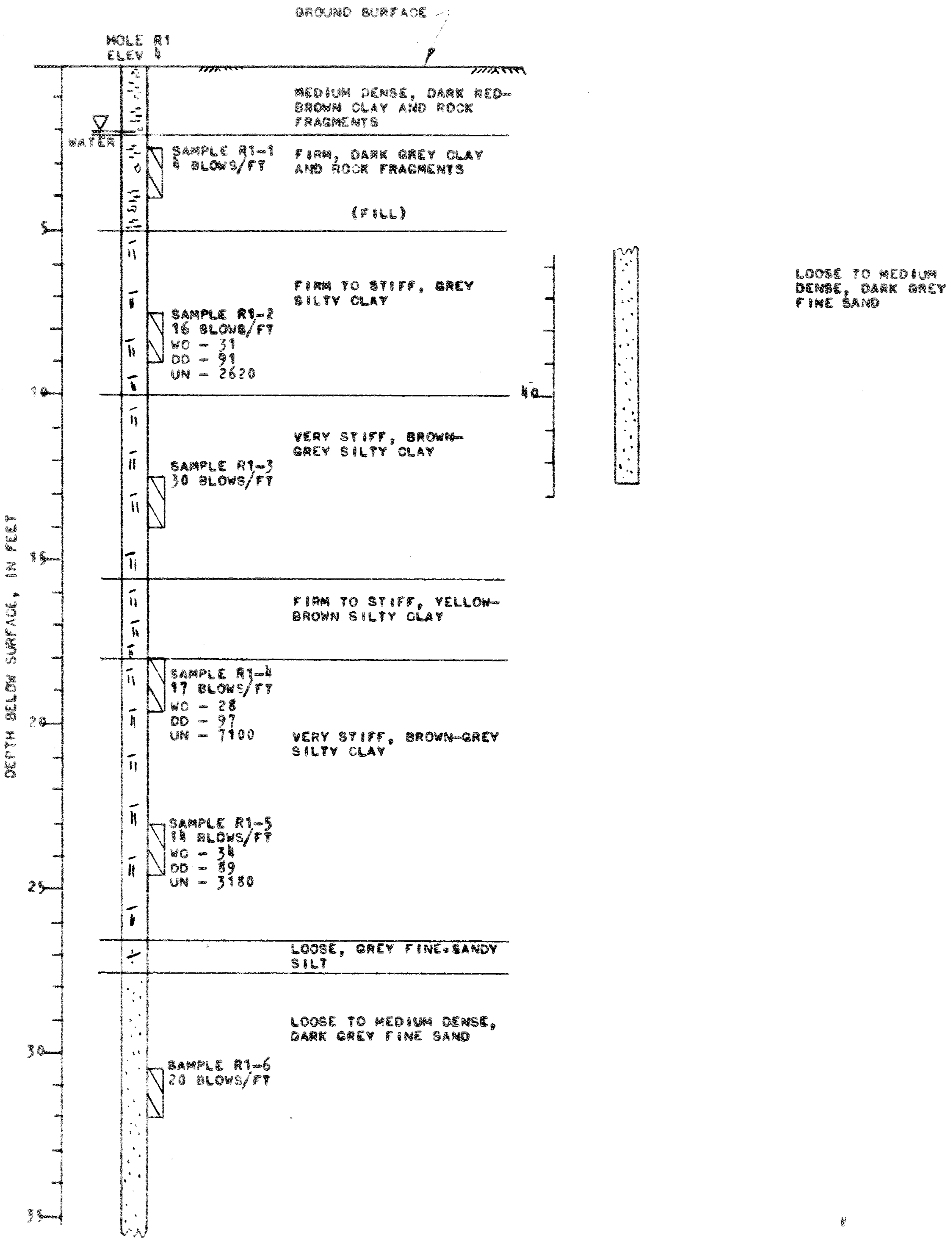
GROUND SURFACE
HOLE R1
ELEV 0
WATER
MEDIUM DENSE, DARK RED-BROWN CLAY AND ROCK FRAGMENTS
SAMPLE R1-1
8 BLOWS/FT
FIRM, DARK GREY CLAY AND ROCK FRAGMENTS
(FILL)
FIRM TO STIFF, GREY SILTY CLAY
SAMPLE R1-2
16 BLOWS/FT
WC - 31
DD - 91
UN - 2620
VERY STIFF, BROWN-GREY SILTY CLAY
SAMPLE R1-3
30 BLOWS/FT
FIRM TO STIFF, YELLOW-BROWN SILTY CLAY
SAMPLE R1-4
17 BLOWS/FT
WC - 28
DD - 97
UN - 7100
VERY STIFF, BROWN-GREY SILTY CLAY
SAMPLE R1-5
14 BLOWS/FT
WC - 34
DD - 89
UN - 3180
LOOSE, GREY FINE SANDY SILT
LOOSE TO MEDIUM DENSE, DARK GREY FINE SAND
SAMPLE R1-6
20 BLOWS/FT
DEPTH BELOW SURFACE, IN FEET
5
10
15
20
25
30
35
10
LOOSE TO MEDIUM DENSE, DARK GREY FINE SAND

FIG. 38 — LOGS OF BORINGS

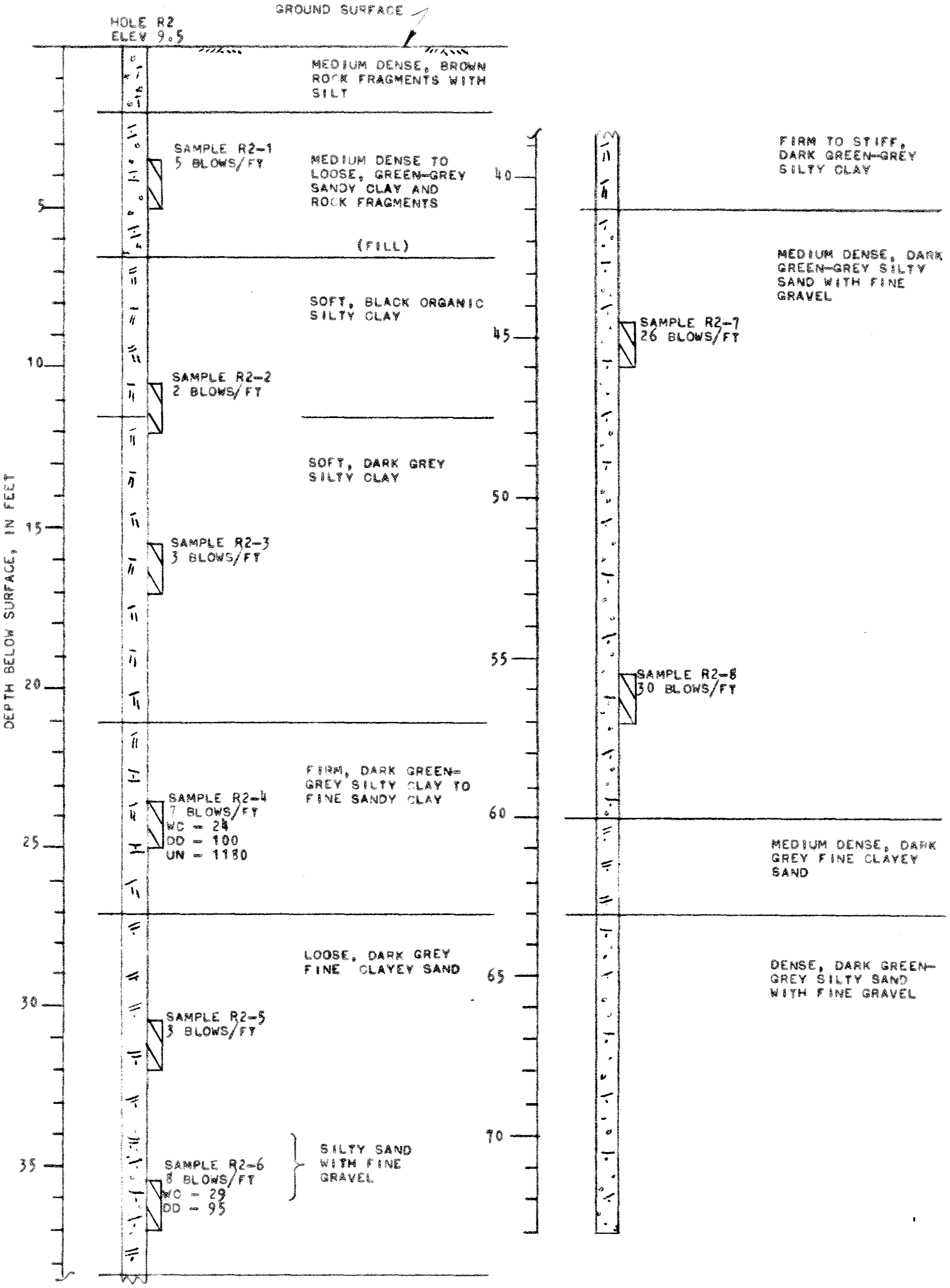
HOLE R2
ELEV 9.5
GROUND SURFACE
MEDIUM DENSE, BROWN ROCK FRAGMENTS WITH SILT
SAMPLE R2-1
5 BLOWS/FT
MEDIUM DENSE TO LOOSE, GREEN-GREY SANDY CLAY AND ROCK FRAGMENTS
(FILL)
SOFT, BLACK ORGANIC SILTY CLAY
SAMPLE R2-2
2 BLOWS/FT
SOFT, DARK GREY SILTY CLAY
SAMPLE R2-3
3 BLOWS/FT
FIRM, DARK GREEN-GREY SILTY CLAY TO FINE SANDY CLAY
SAMPLE R2-4
7 BLOWS/FT
WC = 24
DD = 100
UN = 1180
LOOSE, DARK GREY FINE CLAYEY SAND
SAMPLE R2-5
3 BLOWS/FT
SILTY SAND WITH FINE GRAVEL
SAMPLE R2-6
8 BLOWS/FT
WC = 29
DD = 95
DEPTH BELOW SURFACE, IN FEET
5
10
15
20
25
30
35
40
45
50
55
60
65
70
FIRM TO STIFF, DARK GREEN-GREY SILTY CLAY
MEDIUM DENSE, DARK GREEN-GREY SILTY SAND WITH FINE GRAVEL
SAMPLE R2-7
26 BLOWS/FT
SAMPLE R2-8
30 BLOWS/FT
MEDIUM DENSE, DARK GREY FINE CLAYEY SAND
DENSE, DARK GREEN-GREY SILTY SAND WITH FINE GRAVEL

FIG. 39 – LOGS OF BORINGS

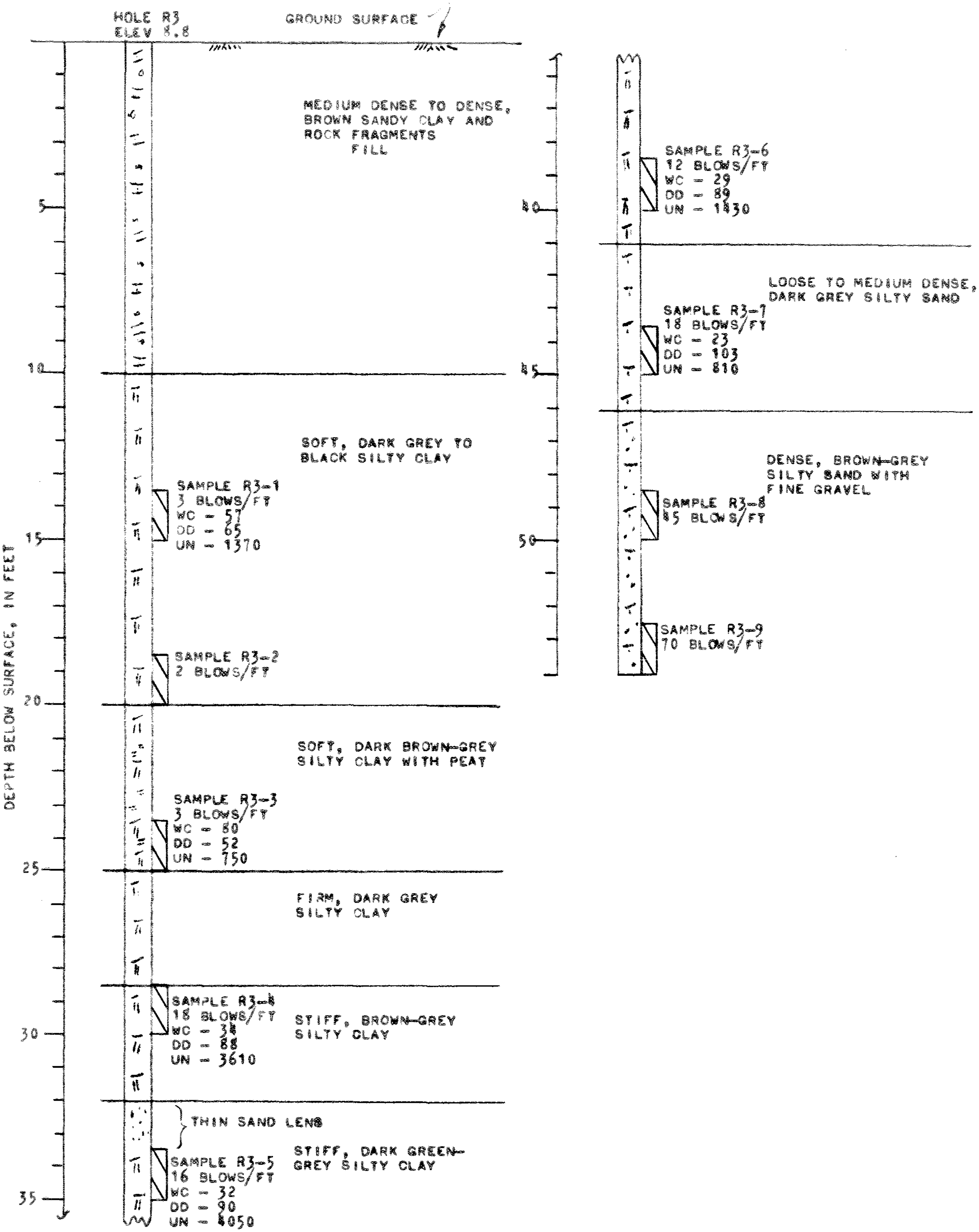

FIG. 40 - LOGS OF BORINGS

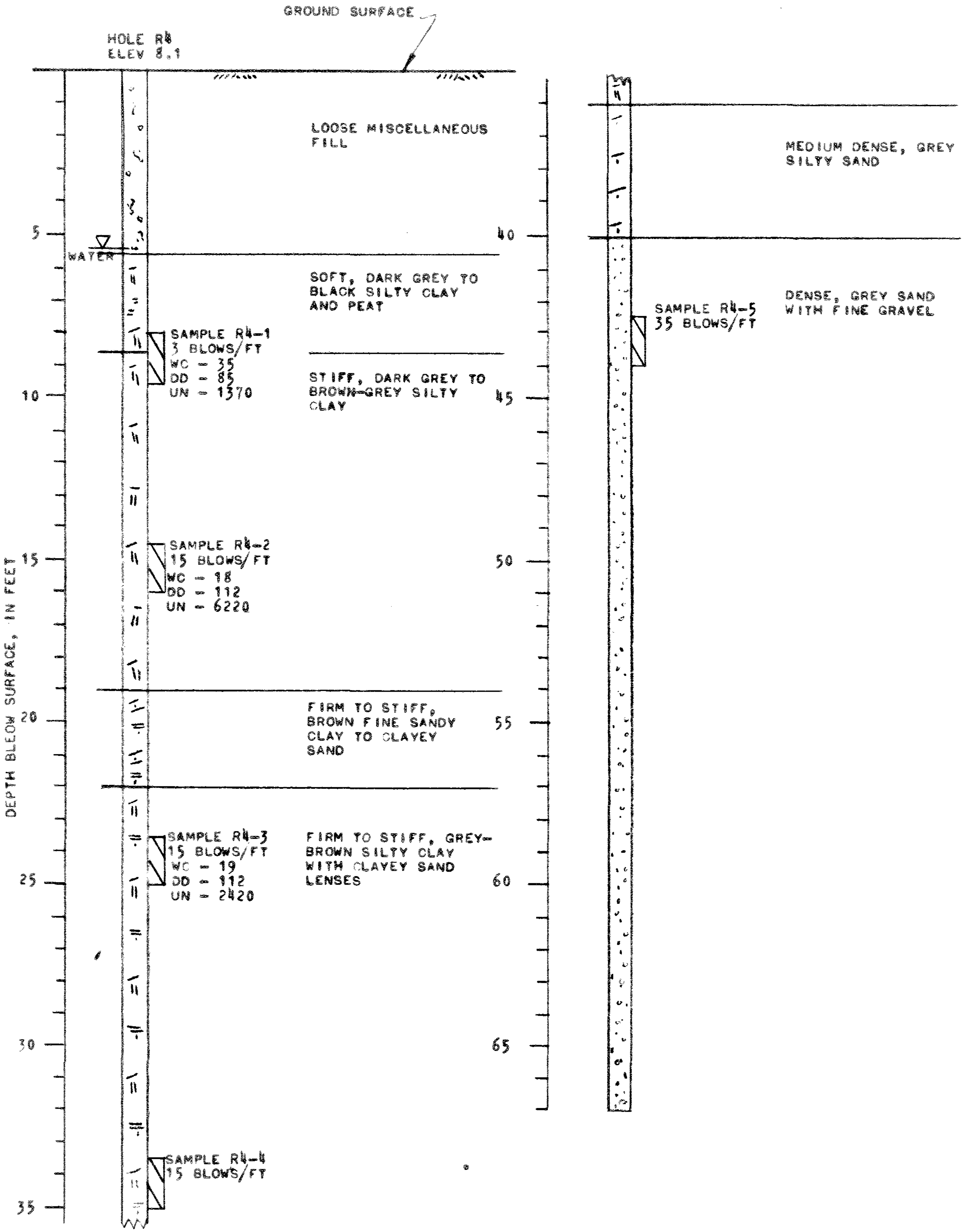
GROUND SURFACE
HOLE R4
ELEV 8.1
WATER
DEPTH BLEOW SURFACE, IN FEET
5
10
15
20
25
30
35
40
45
50
55
60
65
LOOSE MISCELLANEOUS FILL
SOFT, DARK GREY TO BLACK SILTY CLAY AND PEAT
SAMPLE R4-1
3 BLOWS/FT
WC - 35
DD - 85
UN - 1370
STIFF, DARK GREY TO BROWN-GREY SILTY CLAY
SAMPLE R4-2
15 BLOWS/FT
WC - 18
DD - 112
UN - 6220
FIRM TO STIFF, BROWN FINE SANDY CLAY TO CLAYEY SAND
SAMPLE R4-3
15 BLOWS/FT
WC - 19
DD - 112
UN - 2420
FIRM TO STIFF, GREY-BROWN SILTY CLAY WITH CLAYEY SAND LENSES
SAMPLE R4-4
15 BLOWS/FT
MEDIUM DENSE, GREY SILTY SAND
SAMPLE R4-5
35 BLOWS/FT
DENSE, GREY SAND WITH FINE GRAVEL

FIG. 41 - LOGS OF BORINGS

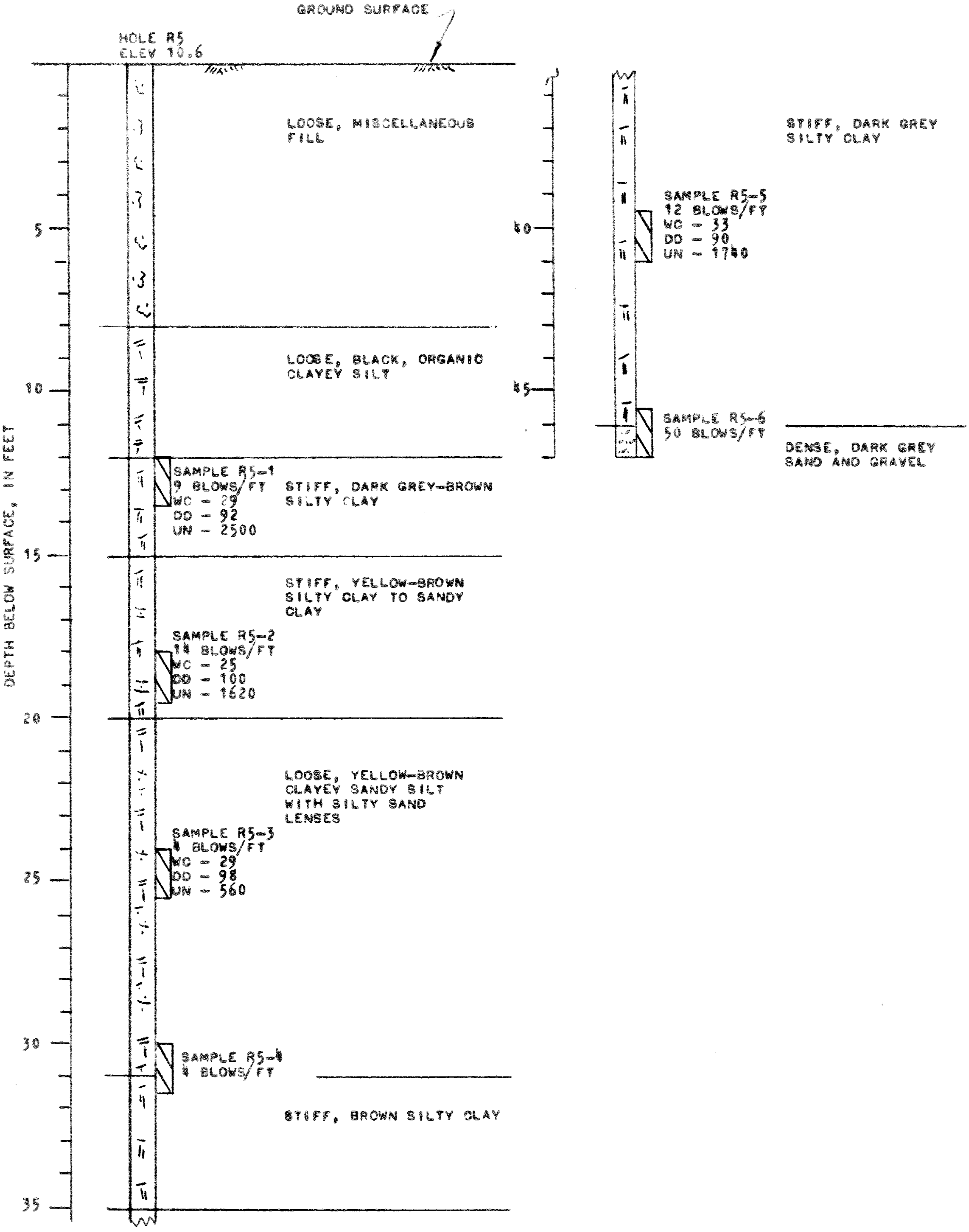

FIG. 42 - LOGS OF BORINGS

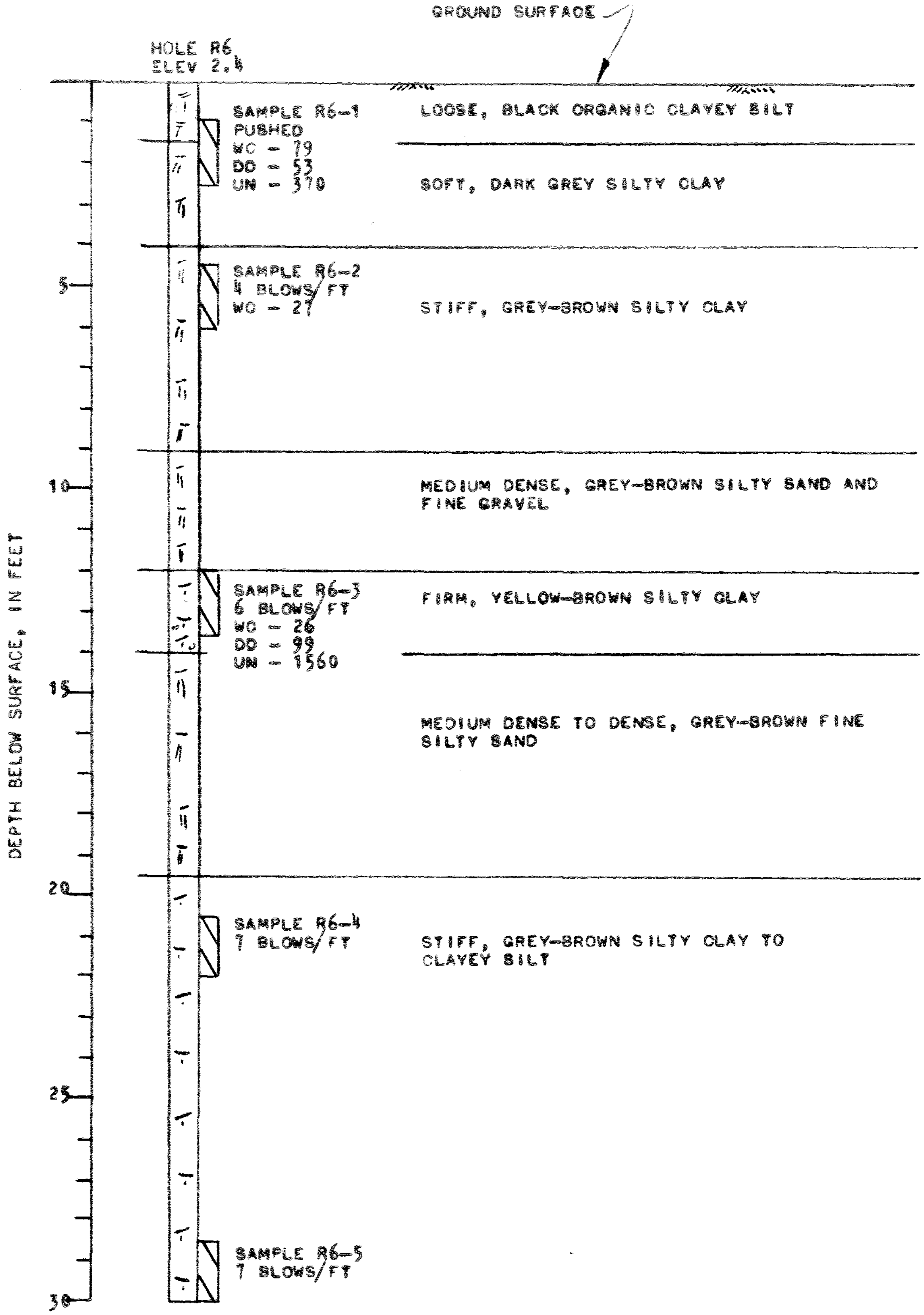
GROUND SURFACE
HOLE R6
ELEV 2.4
DEPTH BELOW SURFACE, IN FEET
5
10
15
20
25
30
SAMPLE R6-1
PUSHED
WC - 79
DD - 53
UN - 370
LOOSE, BLACK ORGANIC CLAYEY SILT
SOFT, DARK GREY SILTY CLAY
SAMPLE R6-2
4 BLOWS/FT
WC - 27
STIFF, GREY-BROWN SILTY CLAY
MEDIUM DENSE, GREY-BROWN SILTY SAND AND FINE GRAVEL
SAMPLE R6-3
6 BLOWS/FT
WC - 26
DD - 99
UN - 1560
FIRM, YELLOW-BROWN SILTY CLAY
MEDIUM DENSE TO DENSE, GREY-BROWN FINE SILTY SAND
SAMPLE R6-4
7 BLOWS/FT
STIFF, GREY-BROWN SILTY CLAY TO CLAYEY SILT
SAMPLE R6-5
7 BLOWS/FT

FIG. 43 – LOGS OF BORINGS

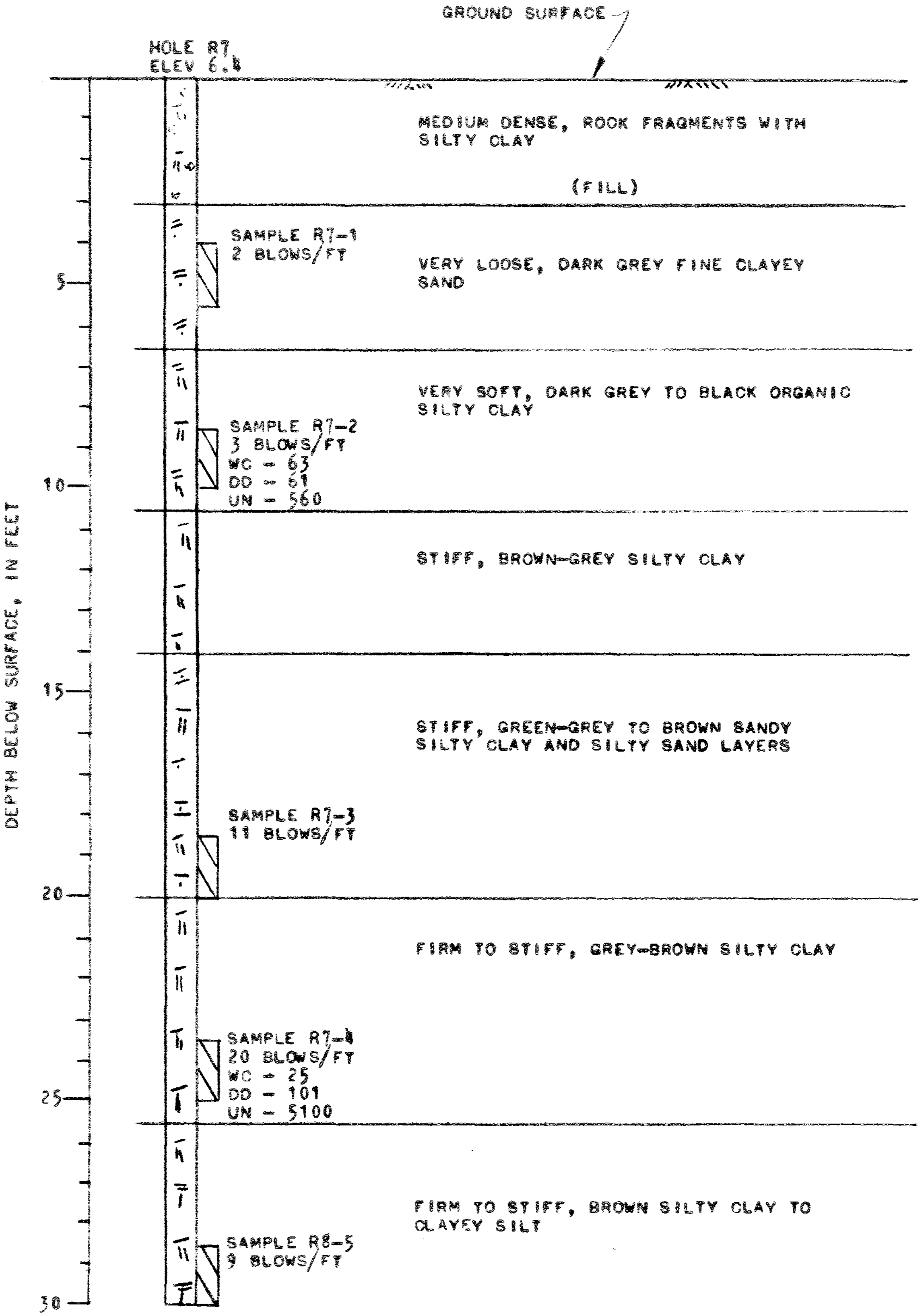
GROUND SURFACE
HOLE R7
ELEV 6.4
MEDIUM DENSE, ROCK FRAGMENTS WITH SILTY CLAY
(FILL)
SAMPLE R7-1
2 BLOWS/FT
VERY LOOSE, DARK GREY FINE CLAYEY SAND
VERY SOFT, DARK GREY TO BLACK ORGANIC SILTY CLAY
SAMPLE R7-2
3 BLOWS/FT
WC - 63
DD - 69
UN - 560
STIFF, BROWN-GREY SILTY CLAY
STIFF, GREEN-GREY TO BROWN SANDY SILTY CLAY AND SILTY SAND LAYERS
SAMPLE R7-3
11 BLOWS/FT
FIRM TO STIFF, GREY-BROWN SILTY CLAY
SAMPLE R7-4
20 BLOWS/FT
WC - 25
DD - 101
UN - 5100
FIRM TO STIFF, BROWN SILTY CLAY TO CLAYEY SILT
SAMPLE R8-5
9 BLOWS/FT
DEPTH BELOW SURFACE, IN FEET
5
10
15
20
25
30

FIG. 44 – LOGS OF BORINGS

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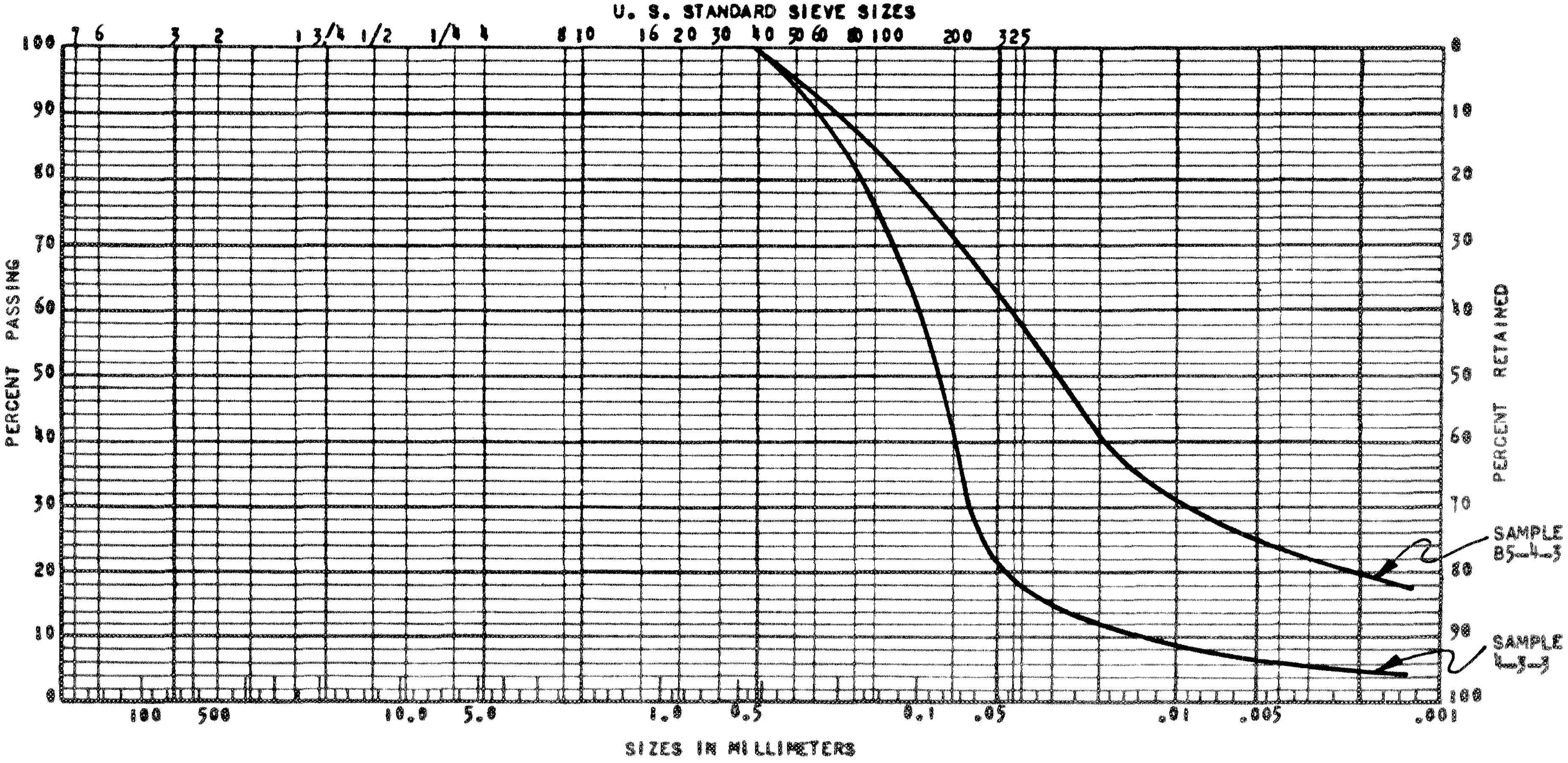
PROJECT: GUADALUPE RIVER	MECHANICAL ANALYSIS CURVES

SAMPLE NO.	DEPTH	LL	PI	CLASSIFICATION	SAMPLE NO.	DEPTH	LL	PI	CLASSIFICATION
4-3-3	12'	-	-	SM	B5-4-3	18'	-	-	CL

UNIFIED SOIL CLASSIFICATION

COBBLES	GRAVEL		SAND			CLAY AND SILT
	COARSE	FINE	COARSE	MEDIUM	FINE	



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PROJECT:
GUADALUPE RIVER

MECHANICAL ANALYSIS CURVES

SAMPLE NO.	DEPTH	LL	PI	CLASSIFICATION	SAMPLE NO.	DEPTH	LL	PI	CLASSIFICATION
8-3-3	15'	-	-	SM	19-3-3	19'	62	27	OH

UNIFIED SOIL CLASSIFICATION

COBBLES	GRAVEL		SAND			CLAY AND SILT
	COARSE	FINE	COARSE	MEDIUM	FINE	

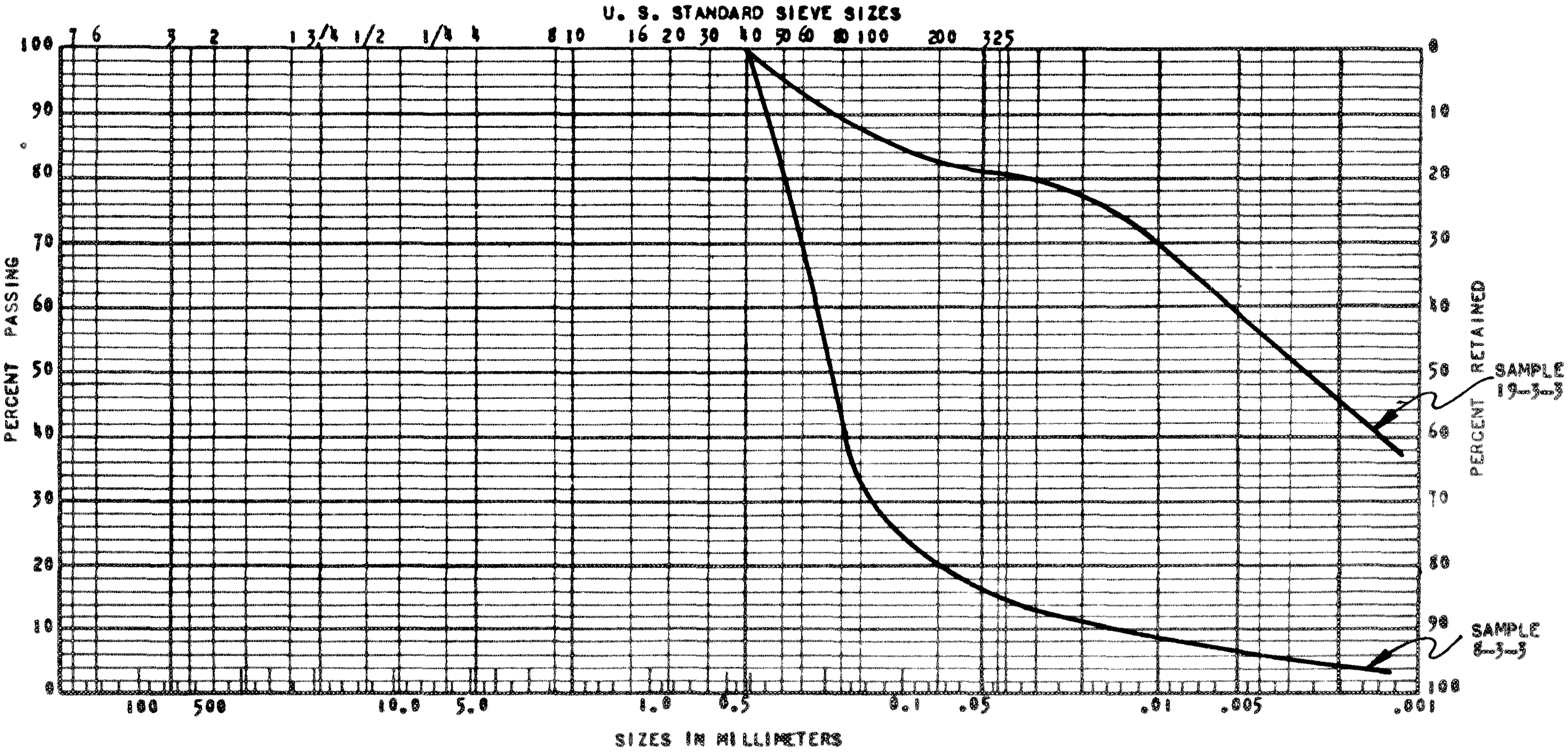

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FIG. 86

% 9646

PROJECT: GUADALUPE RIVER	MECHANICAL ANALYSIS CURVES

SAMPLE NO.	DEPTH	LL	PI	CLASSIFICATION	SAMPLE NO.	DEPTH	LL	PI	CLASSIFICATION
17-3-3	13'	-	-	SM	22-2-3	8'	-	-	OH

UNIFIED SOIL CLASSIFICATION

COBBLES	GRAVEL		SAND			CLAY AND SILT
	COARSE	FINE	COARSE	MEDIUM	FINE	

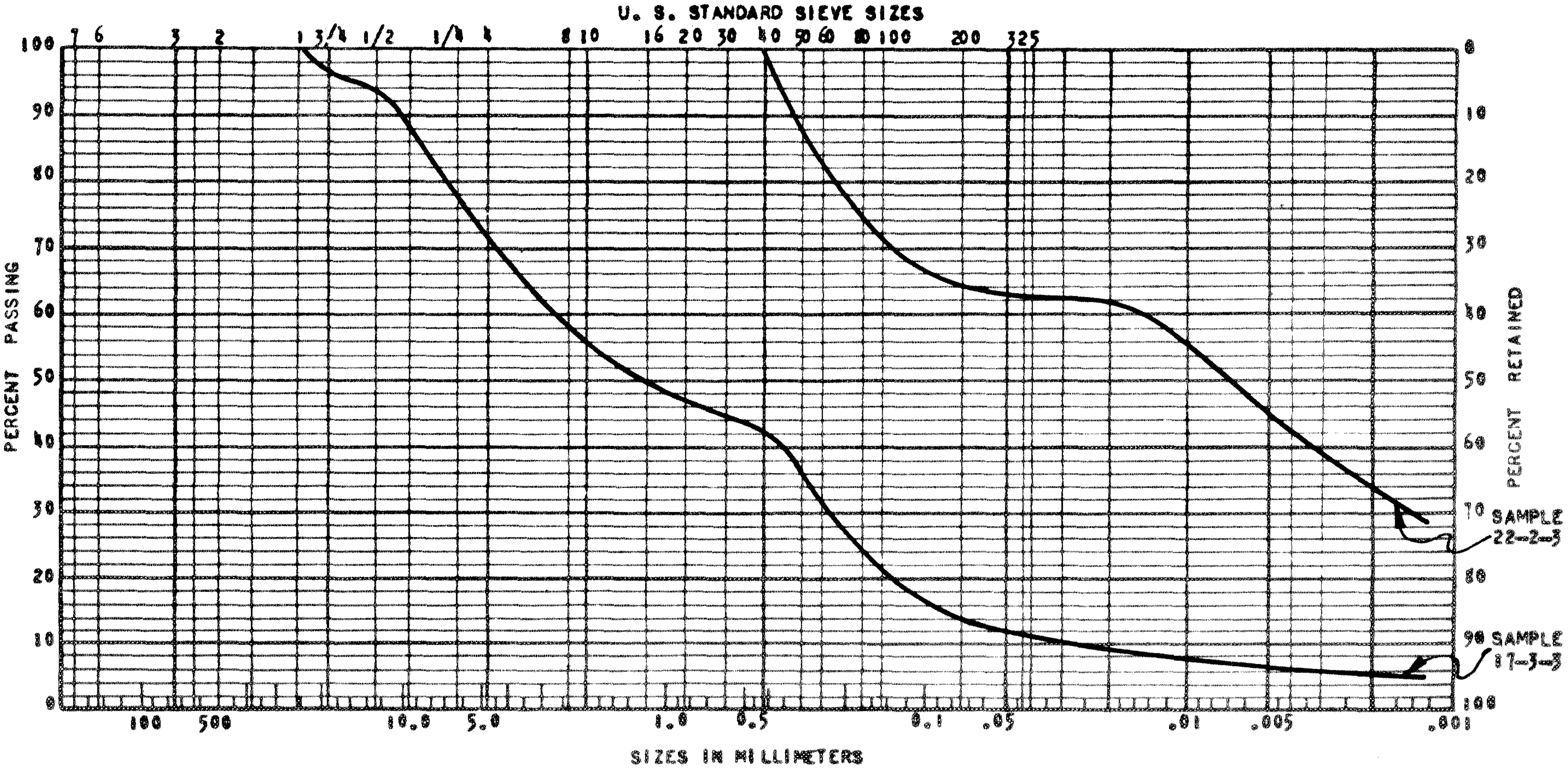

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FIG. 17

PROJECT:
GUADALUPE RIVER

MECHANICAL ANALYSIS CURVES

SAMPLE NO.	DEPTH	LL	PI	CLASSIFICATION	SAMPLE NO.	DEPTH	LL	PI	CLASSIFICATION
SACK SAMPLE #1 (HOLE 5)	0–10'	45	17	ML	SACK SAMPLE #2 (HOLE 6)	0–10'	–	–	ML – CL

UNIFIED SOIL CLASSIFICATION

COBBLES	GRAVEL		SAND			CLAY AND SILT
	COARSE	FINE	COARSE	MEDIUM	FINE	

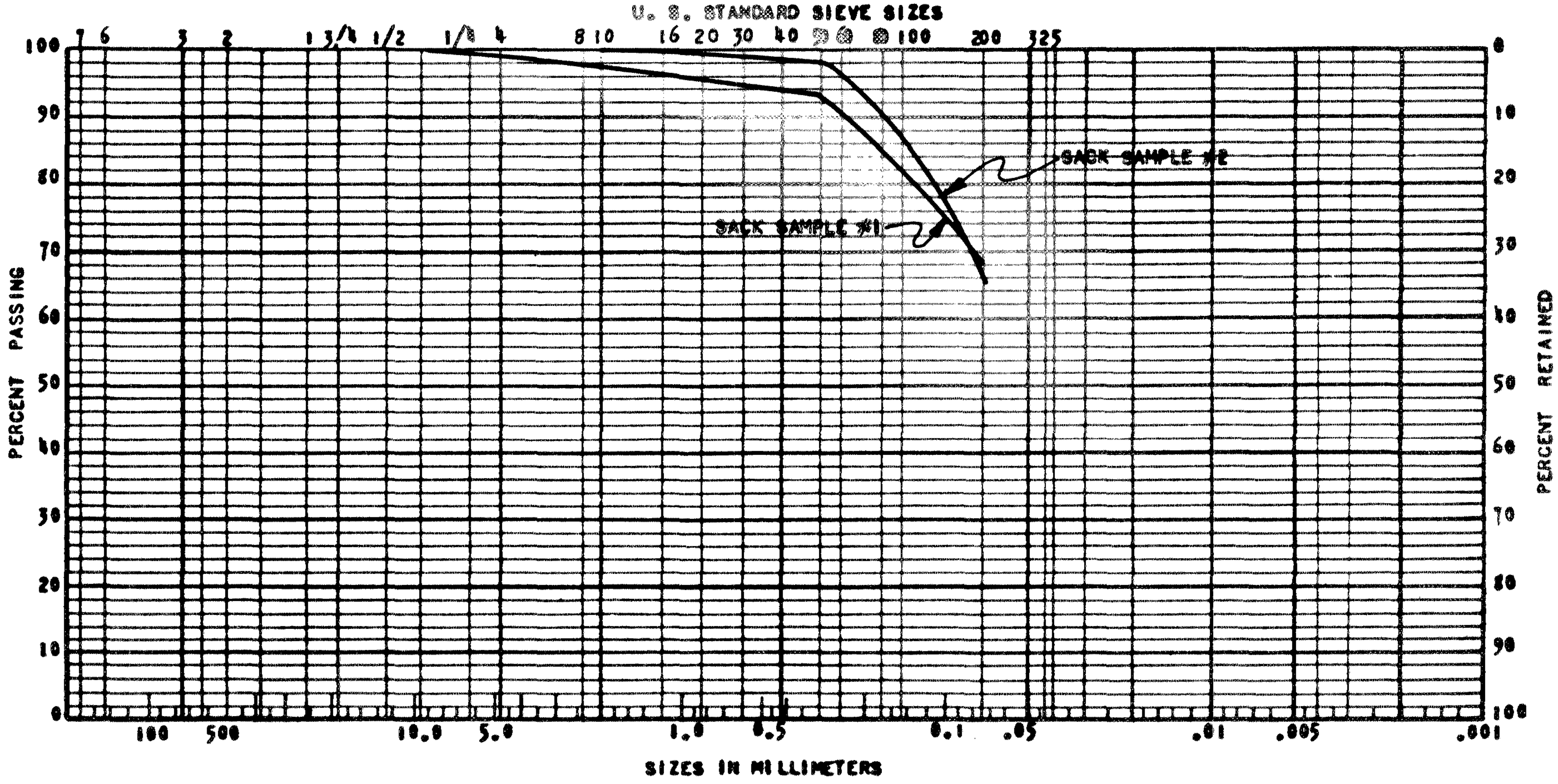


FIG. 18

PROJECT: GUADALUPE RIVER	MECHANICAL ANALYSIS CURVES

SAMPLE NO.	DEPTH	LL	PI	CLASSIFICATION	SAMPLE NO.	DEPTH	LL	PI	CLASSIFICATION
SACK SAMPLE #3 (HOLE 9)	0-10'	34	10	CL - ML	SACK SAMPLE #4 (HOLE 11)	0-10'	56	27	CH

UNIFIED SOIL CLASSIFICATION

COBBLES	GRAVEL		SAND			CLAY AND SILT
	COARSE	FINE	COARSE	MEDIUM	FINE	

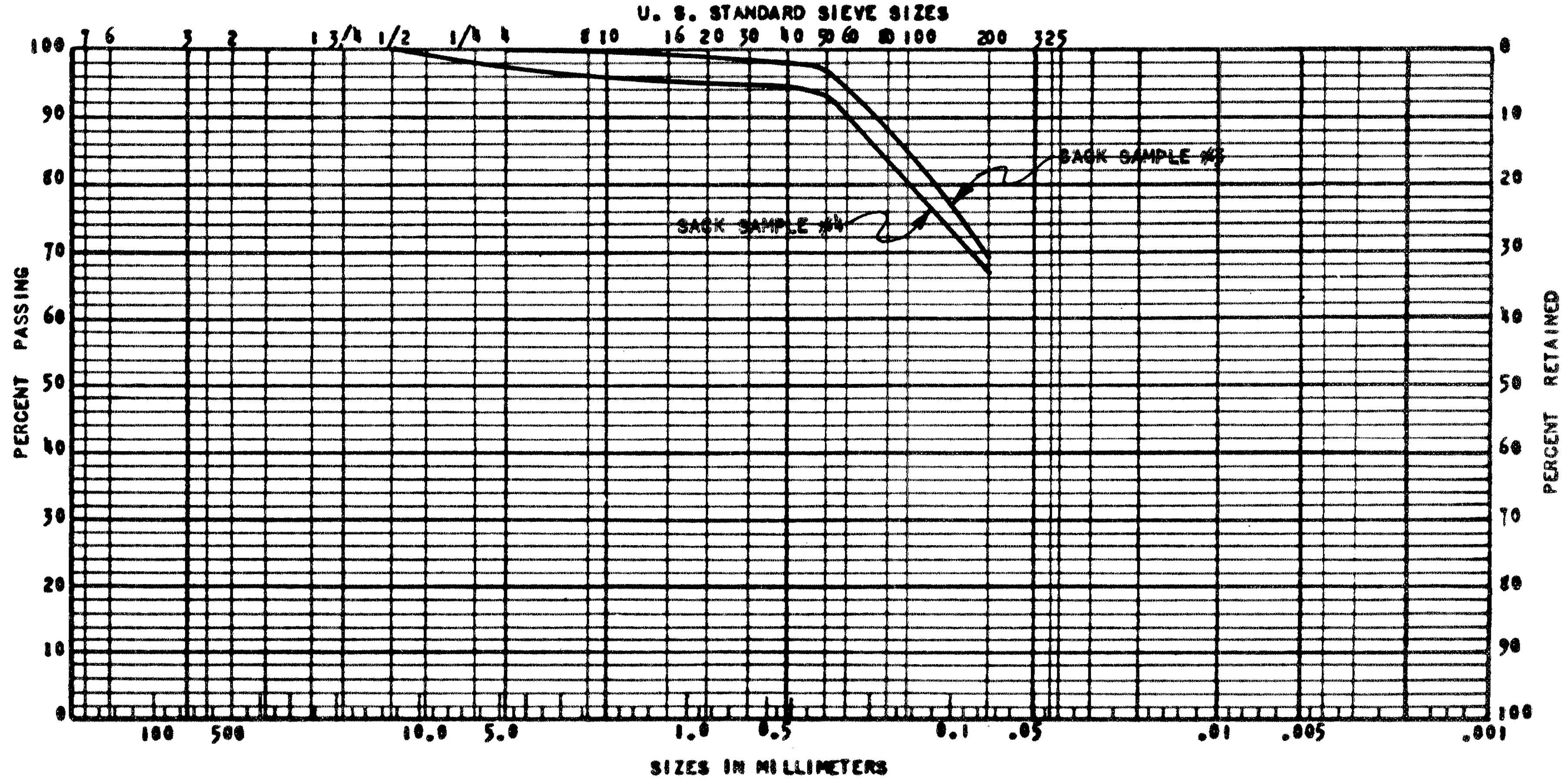


FIG. 19

PROJECT: GUADALUPE RIVER	MECHANICAL ANALYSIS CURVES

SAMPLE NO.	DEPTH	LL	PI	CLASSIFICATION	SAMPLE NO.	DEPTH	LL	PI	CLASSIFICATION
SACK SAMPLE #5 (HOLE 19)	0–10'	64	32	CH	SACK SAMPLE #6 (HOLE R-1)	0–10'	45	24	CL

UNIFIED SOIL CLASSIFICATION

COBBLES	GRAVEL		SAND			CLAY AND SILT
	COARSE	FINE	COARSE	MEDIUM	FINE	

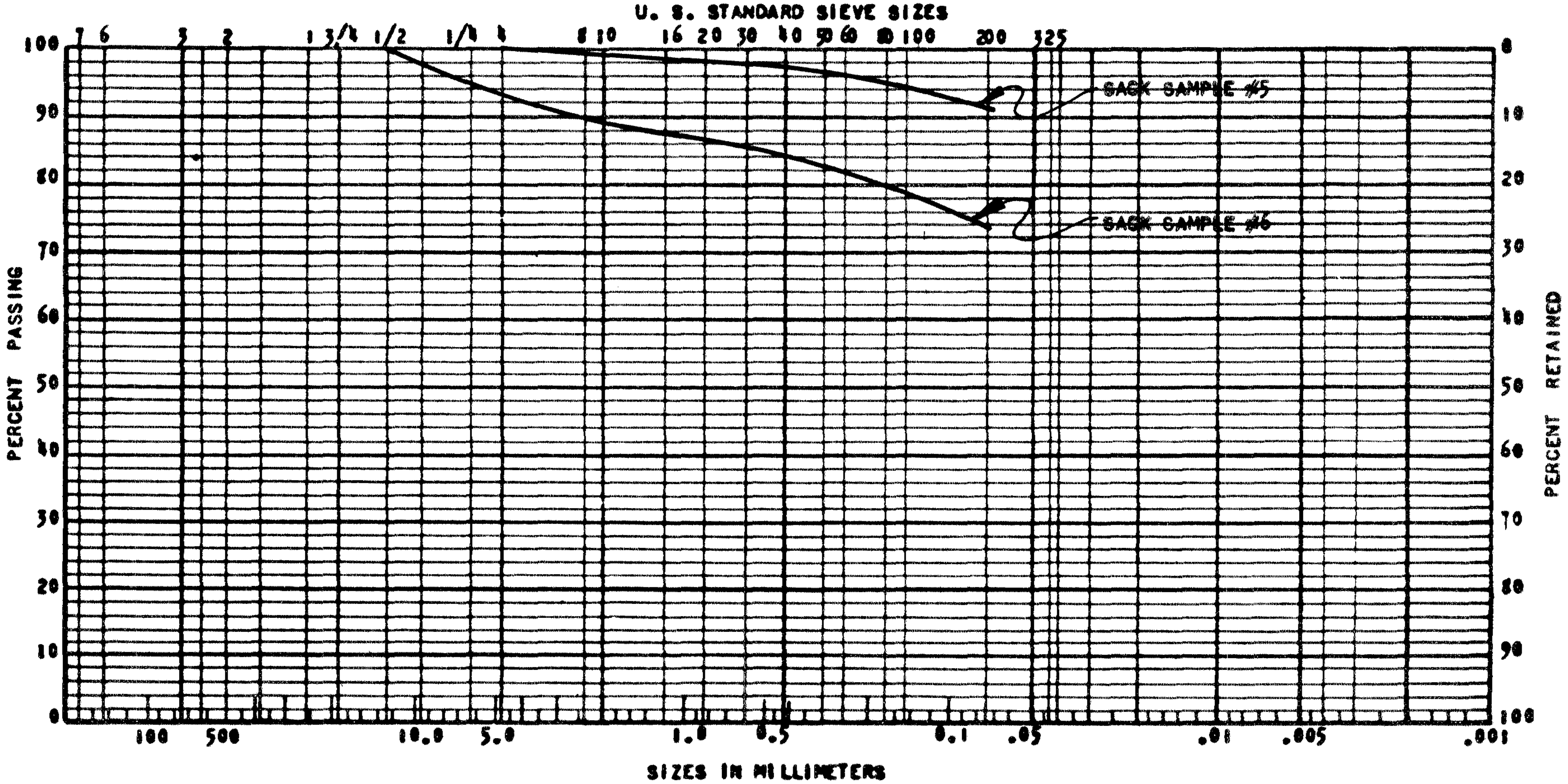


FIG. 50

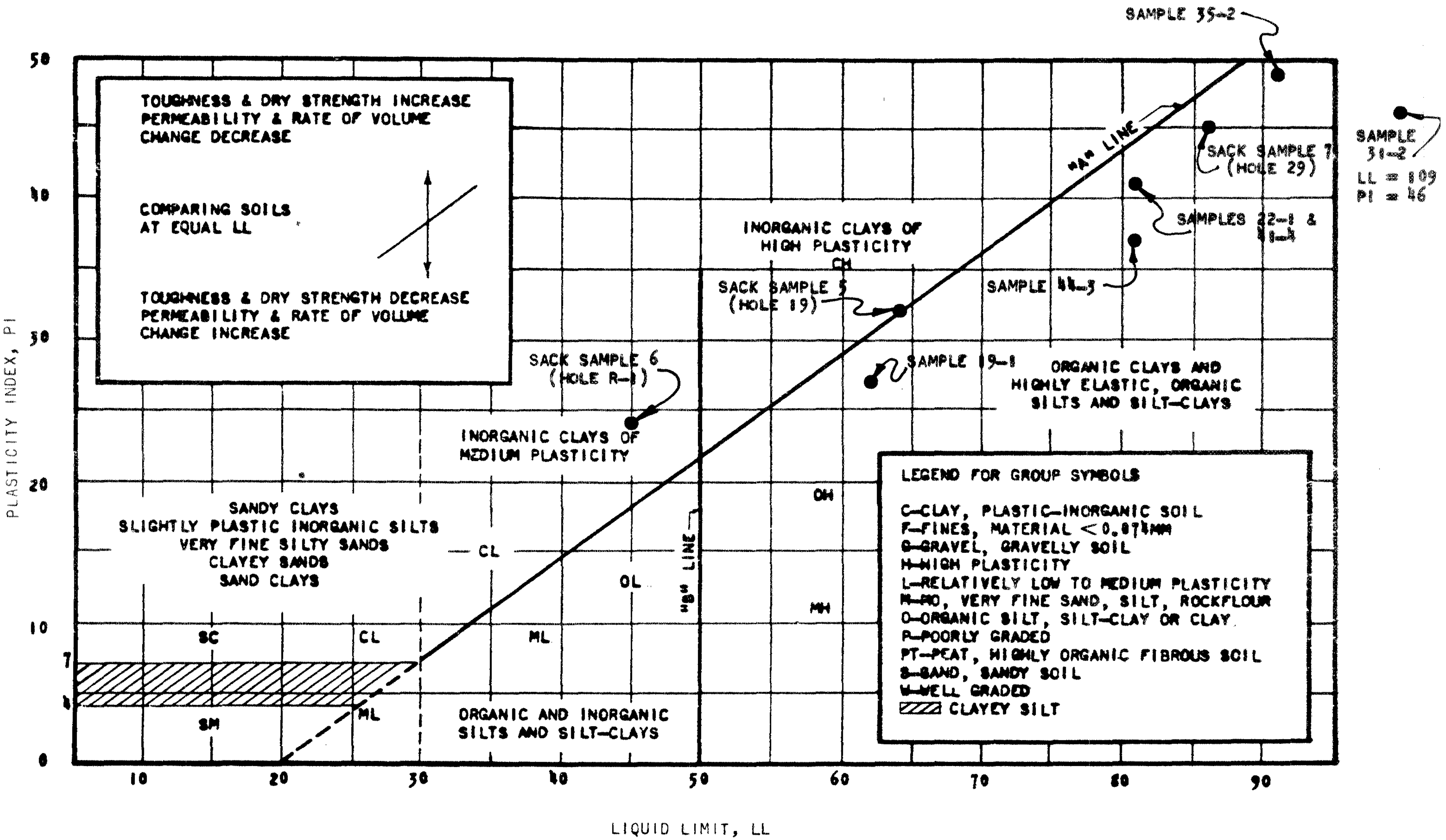


FIG. 51 -PLASTICITY CHART
(REACH #1)

4246

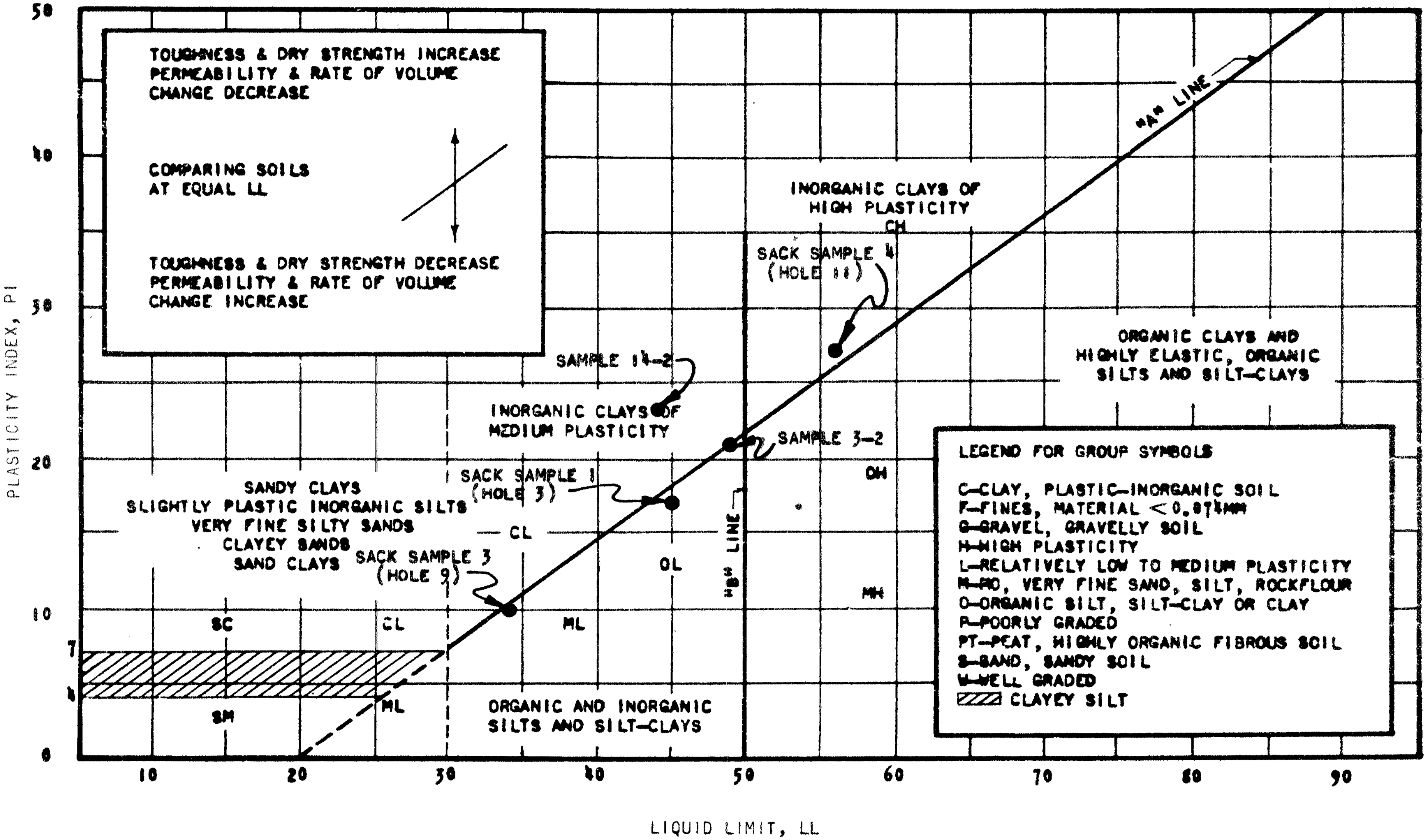
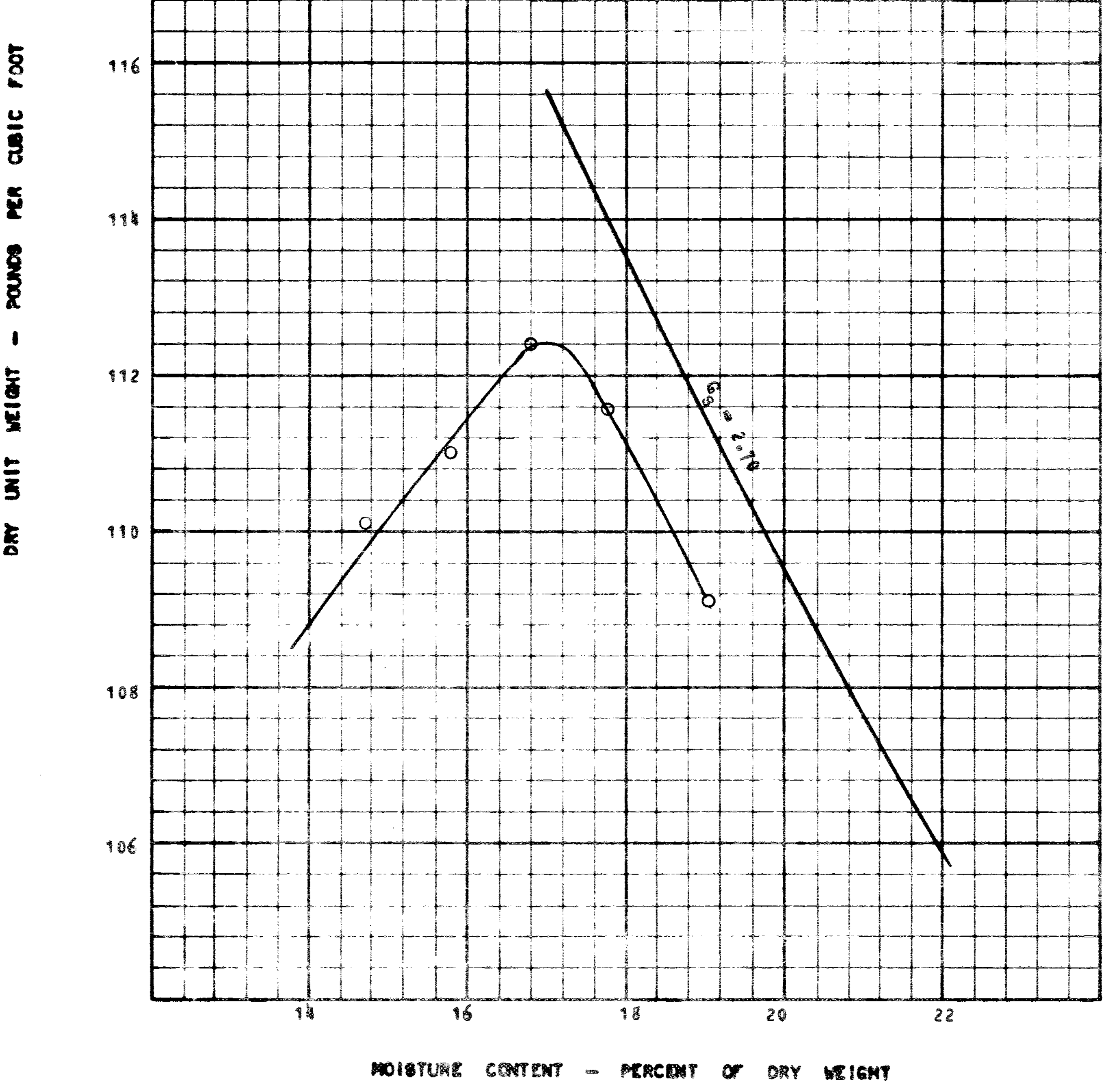


FIG. 52 -PLASTICITY CHART
(REACH #2)

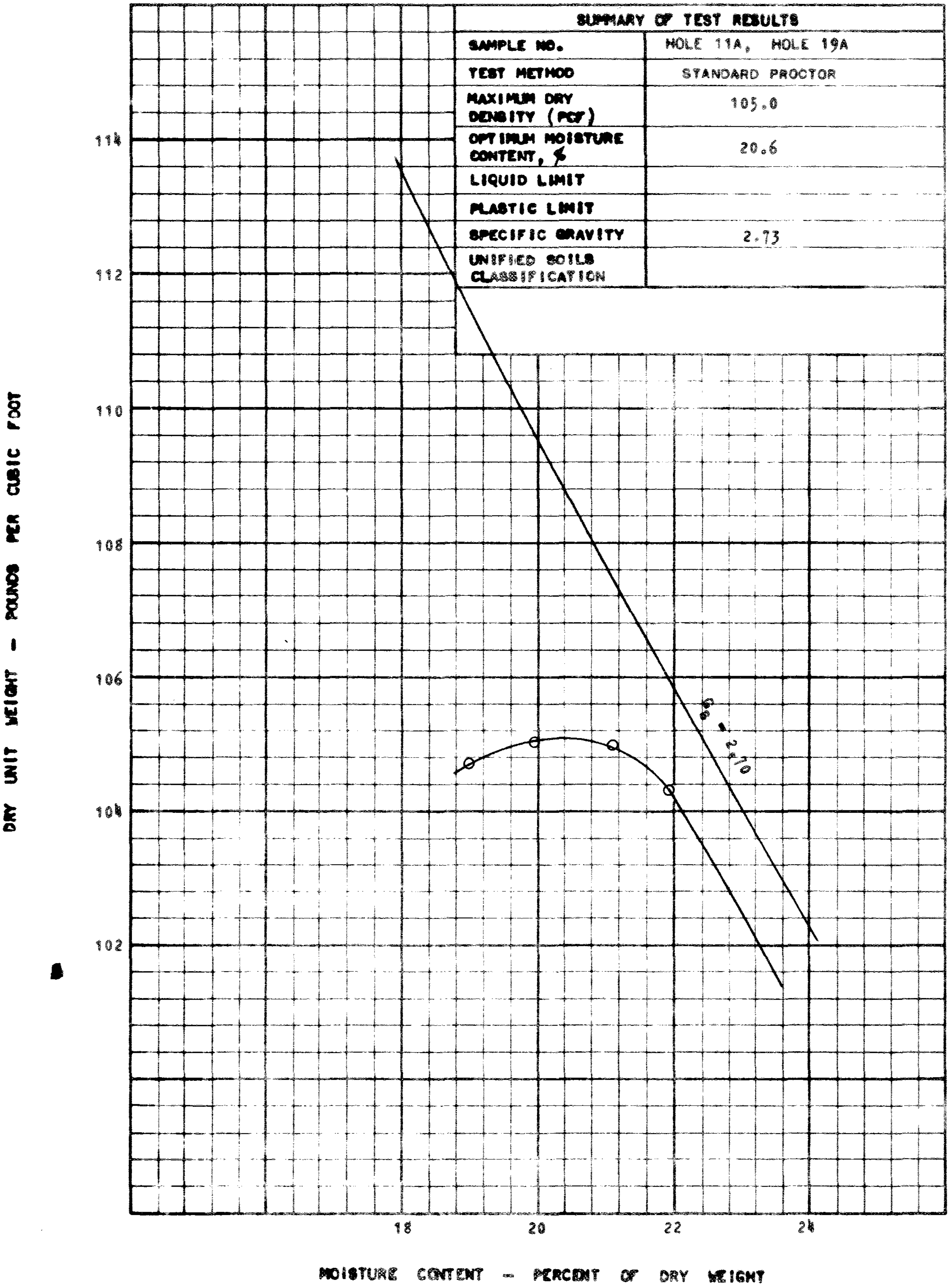
FIG. 52

SUMMARY OF TEST RESULTS	
SAMPLE NO.	HOLE 3A, HOLE 9A
TEST METHOD	STANDARD PROCTOR
MAXIMUM DRY DENSITY (PCF)	112.4
OPTIMUM MOISTURE CONTENT, %	17.0
LIQUID LIMIT	
PLASTIC LIMIT	
SPECIFIC GRAVITY	2.70
UNIFIED SOILS CLASSIFICATION	



COMPACTION TESTS

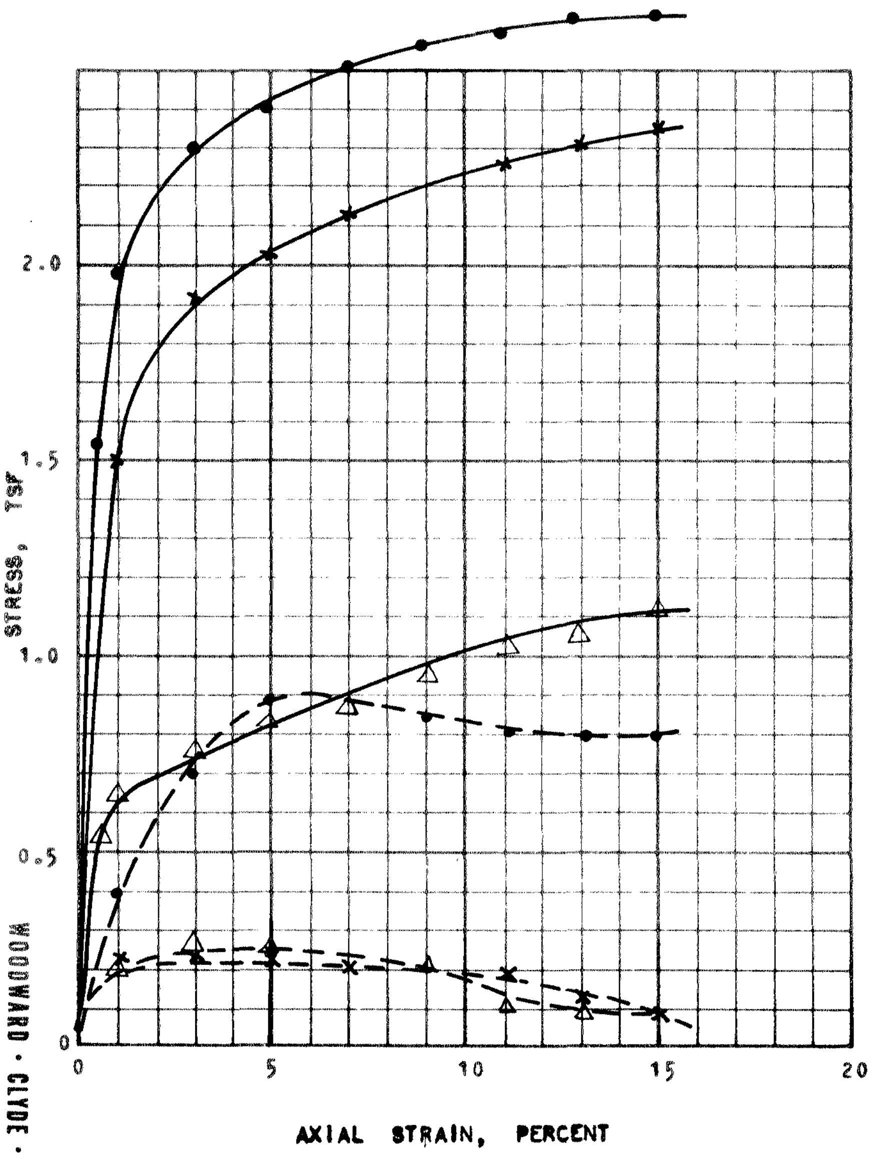
FIG. 53



SUMMARY OF TEST RESULTS	
SAMPLE NO.	HOLE 11A, HOLE 19A
TEST METHOD	STANDARD PROCTOR
MAXIMUM DRY DENSITY (PCF)	105.0
OPTIMUM MOISTURE CONTENT, %	20.6
LIQUID LIMIT	
PLASTIC LIMIT	
SPECIFIC GRAVITY	2.73
UNIFIED SOILS CLASSIFICATION	

COMPACTION TESTS

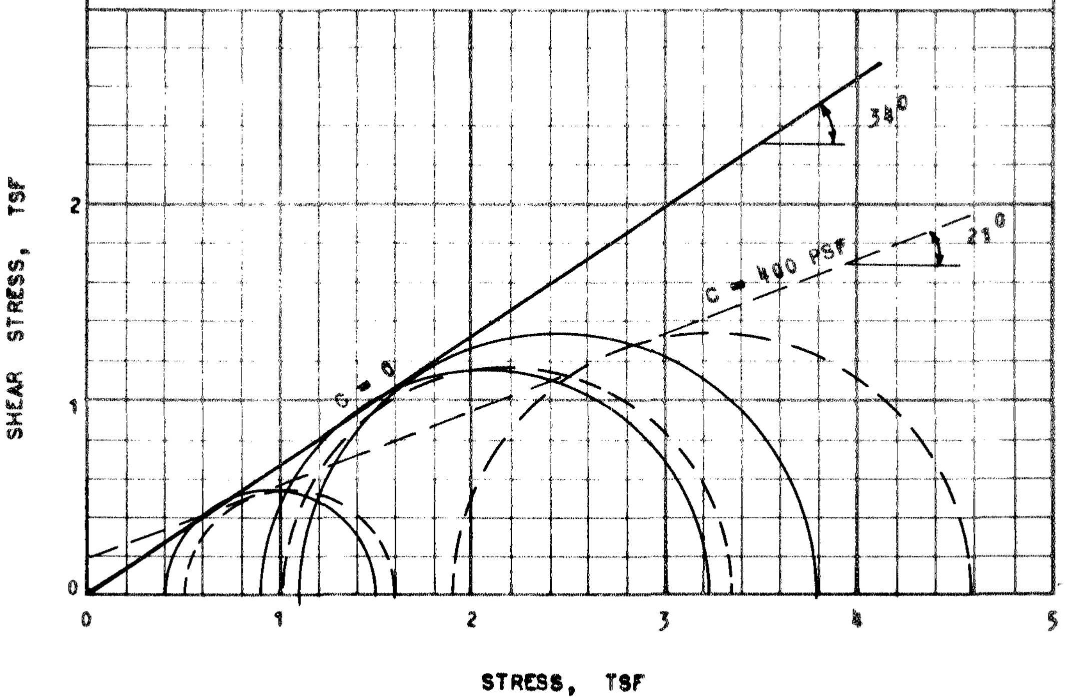
FIG. 54



DEVIATOR STRESS, $\sigma_1 - \sigma_3$ – SOLID LINES

PORE WATER PRESSURE, U – DASH LINES

SPECIMEN NO.	INITIAL MOISTURE CONTENT (%)	INITIAL DRY DENSITY (PCF)	CONFINING PRESSURE, (TSF)	DEVIATOR STRESS AT FAILURE (TSF)	PORE PRESSURE AT FAILURE (TSF)
● A-1	17.1	106.8	1.90	2.69	0.80
✕ A-3	17.0	106.8	1.00	2.35	0.10
△ A-2	16.9	106.8	0.50	1.10	0.10



TYPE OF SPECIMEN : DISTURBED

TYPE OF TEST : C – U

EFFECTIVE STRESS – SOLID LINES

TOTAL STRESS – DASH LINES

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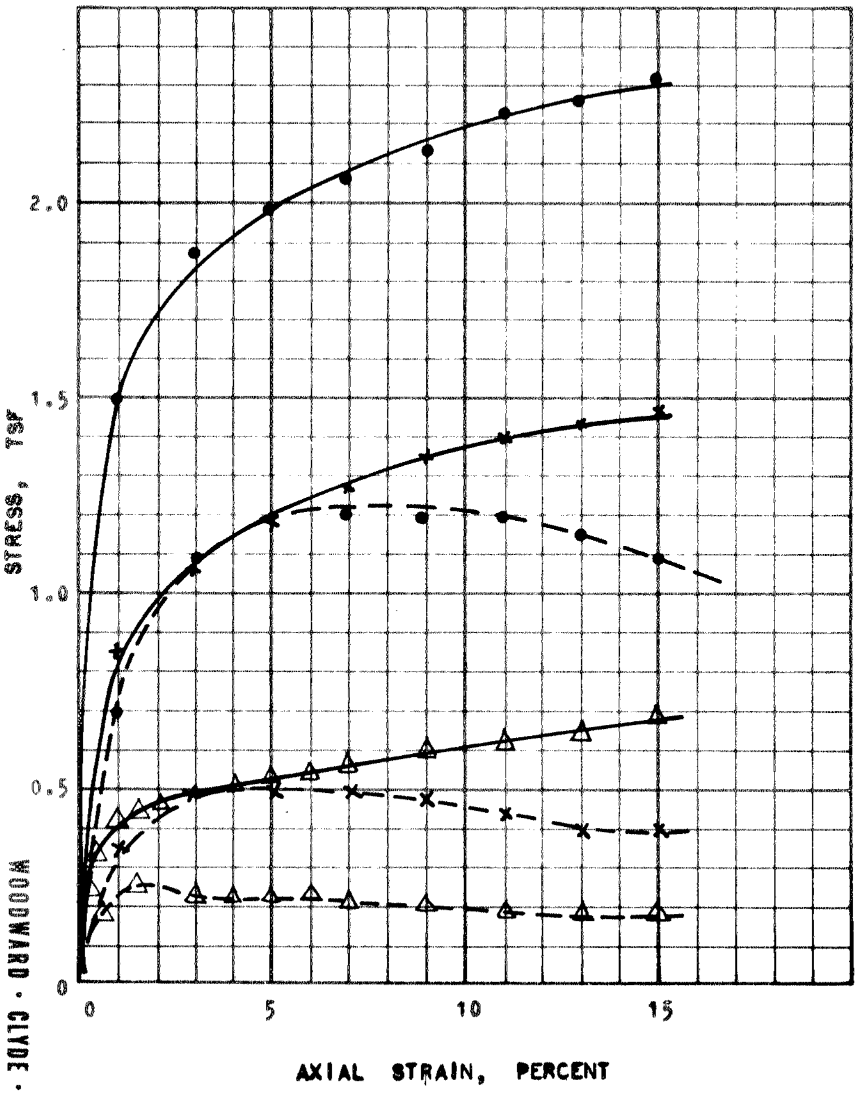
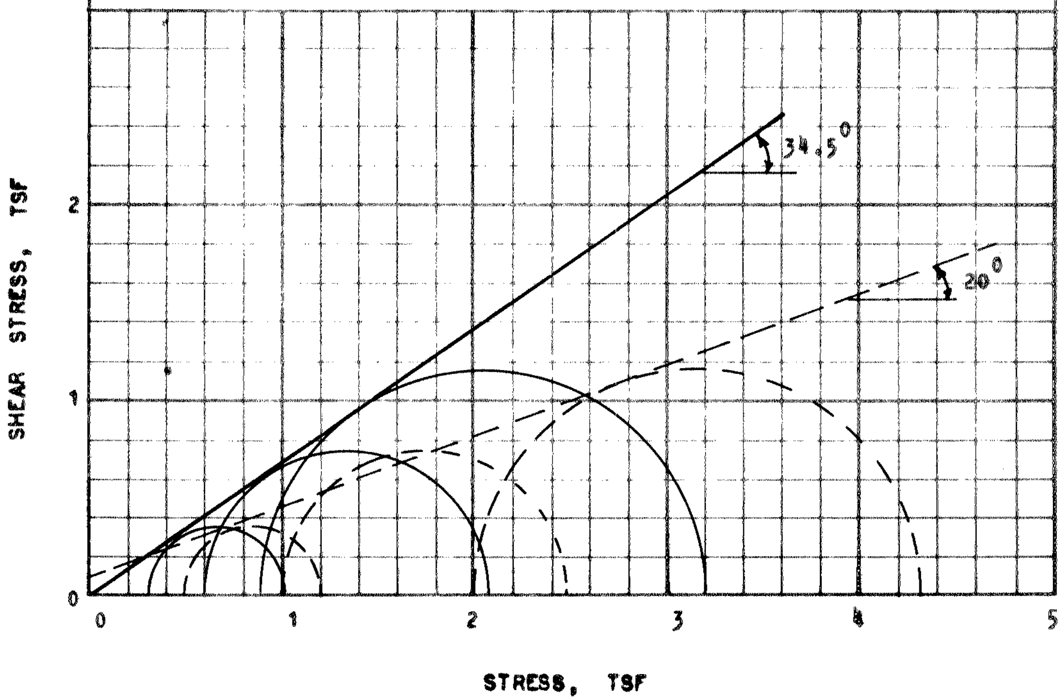
FIG. 55



DEVIATOR STRESS, $\sigma_1 - \sigma_3$ – SOLID LINES

PORE WATER PRESSURE, U – DASH LINES

SPECIMEN NO.	INITIAL MOISTURE CONTENT (%)	INITIAL DRY DENSITY (PCF)	CONFINING PRESSURE, (TSF)	DEVIATOR STRESS AT FAILURE (TSF)	PORE PRESSURE AT FAILURE (TSF)
● B-2	20.2	100.0	2.00	2.30	1.10
✱ B-1	19.5	100.5	1.00	1.48	0.40
△ B-3	20.2	100.0	0.50	0.70	0.20



TYPE OF SPECIMEN : DISTURBED

TYPE OF TEST : C – U

EFFECTIVE STRESS – SOLID LINES

TOTAL STRESS – DASH LINES

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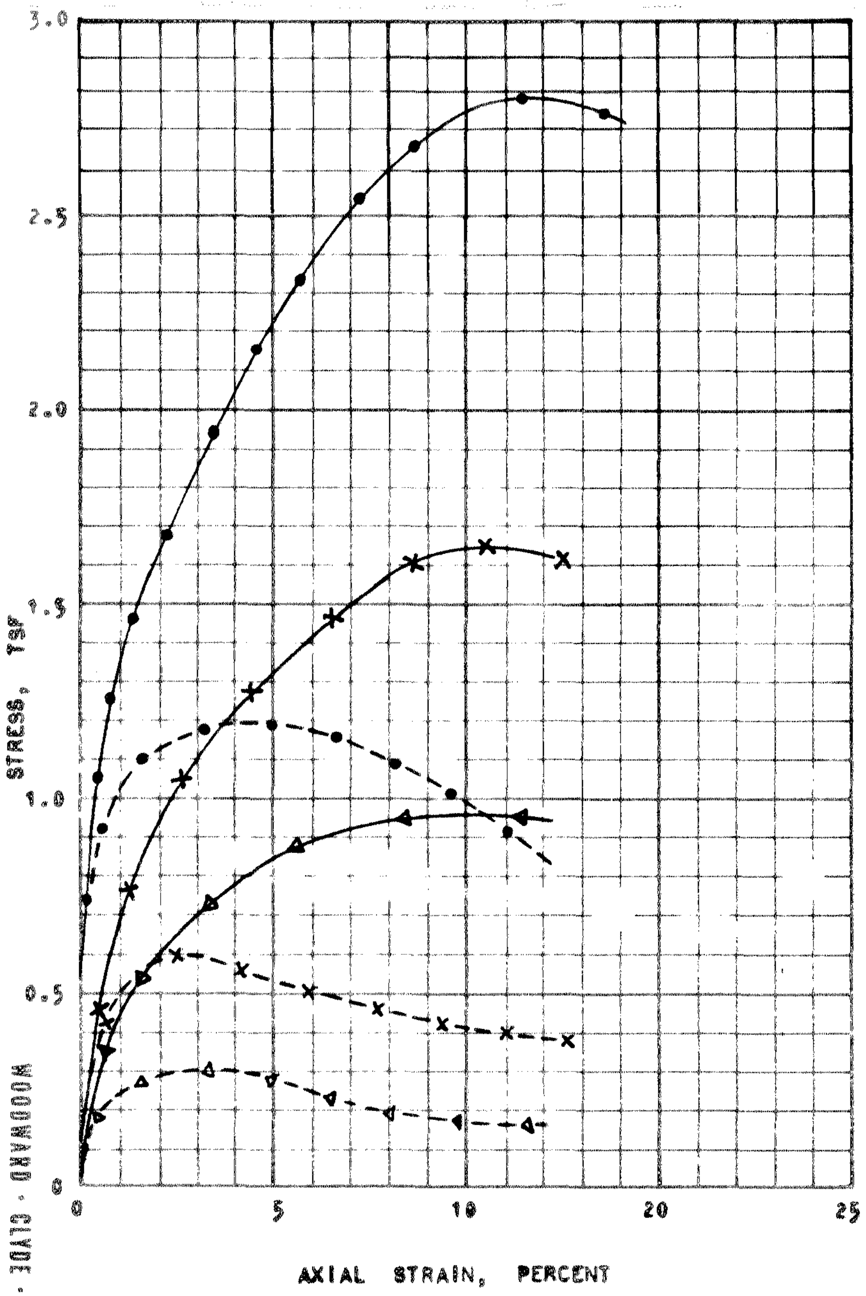
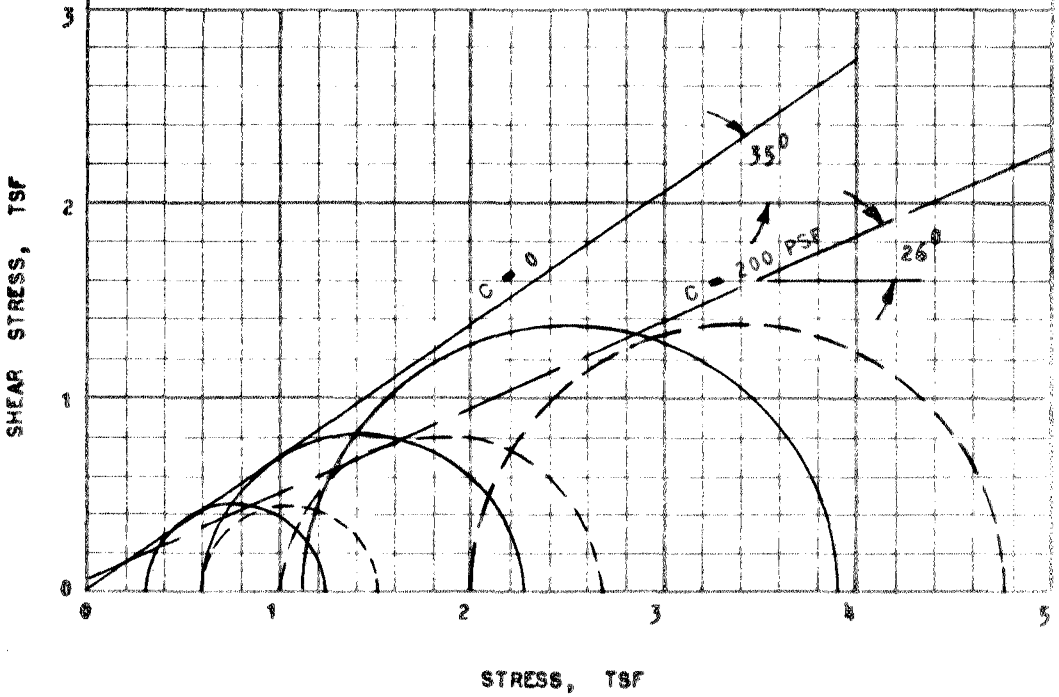
FIG. 56



DEVIATOR STRESS, $\sigma_1 - \sigma_3$ – SOLID LINES

PORE WATER PRESSURE, U – DASH LINES

SPECIMEN NO.	INITIAL MOISTURE CONTENT (%)	INITIAL DRY DENSITY (PCF)	CONFINING PRESSURE, (TSF)	DEVIATOR STRESS AT FAILURE (TSF)	PORE PRESSURE AT FAILURE (TSF)
—●— 7-2-2	33	90	2.00	2.80	0.90
—×— 7-2-3	33	89	1.00	1.63	0.80
—△— 7-2-1	33	89	0.50	0.95	0.20



TYPE OF SPECIMEN : UNDISTURBED   EFFECTIVE STRESS – SOLID LINES

TYPE OF TEST : C–U   TOTAL STRESS – DASH LINES

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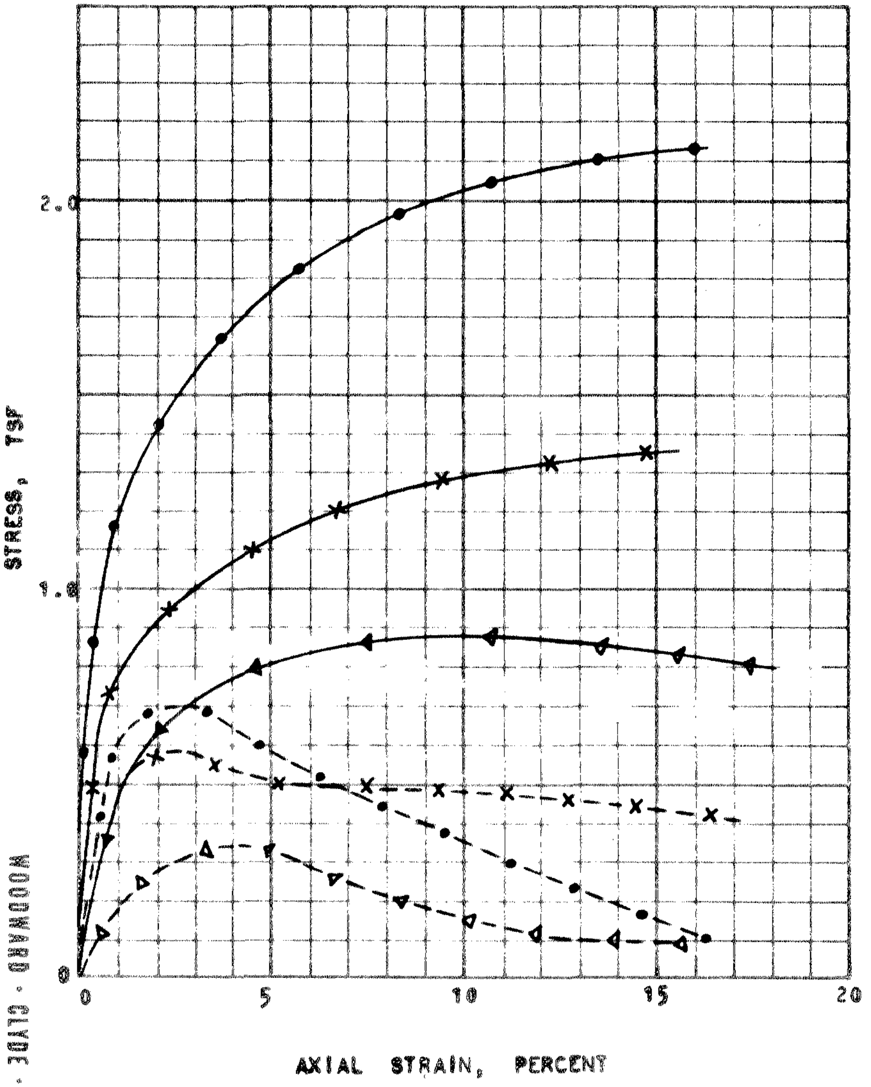
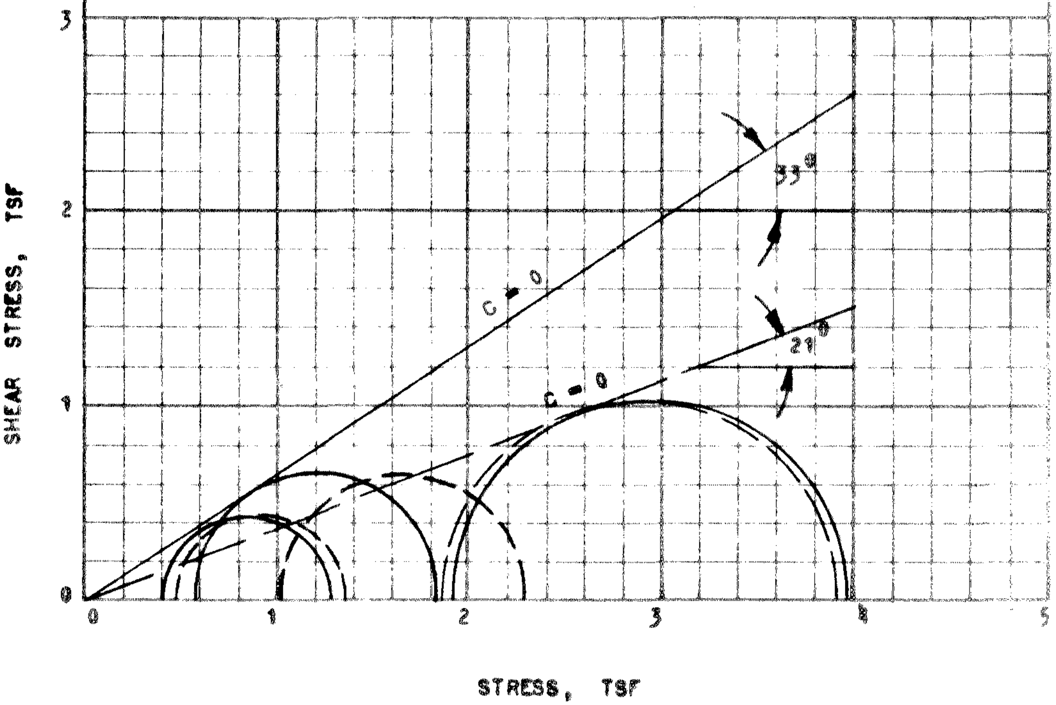
FIG. 57



DEVIATOR STRESS, $\sigma_1 - \sigma_3$ – SOLID LINES

PORE WATER PRESSURE, U – DASH LINES

SPECIMEN NO.	INITIAL MOISTURE CONTENT (%)	INITIAL DRY DENSITY (PCF)	CONFINING PRESSURE, (TSF)	DEVIATOR STRESS AT FAILURE (TSF)	PORE PRESSURE AT FAILURE (TSF)
● 11-3-3	26	100	1.90	2.10	0.15
× 11-3-4	23	104	1.00	1.35	0.45
△ 11-3-2	33	90	0.50	0.85	0.10



TYPE OF SPECIMEN : UNDISTURBED EFFECTIVE STRESS – SOLID LINES

TYPE OF TEST : C-U TOTAL STRESS – DASH LINES

TRIAXIAL COMPRESSION TEST

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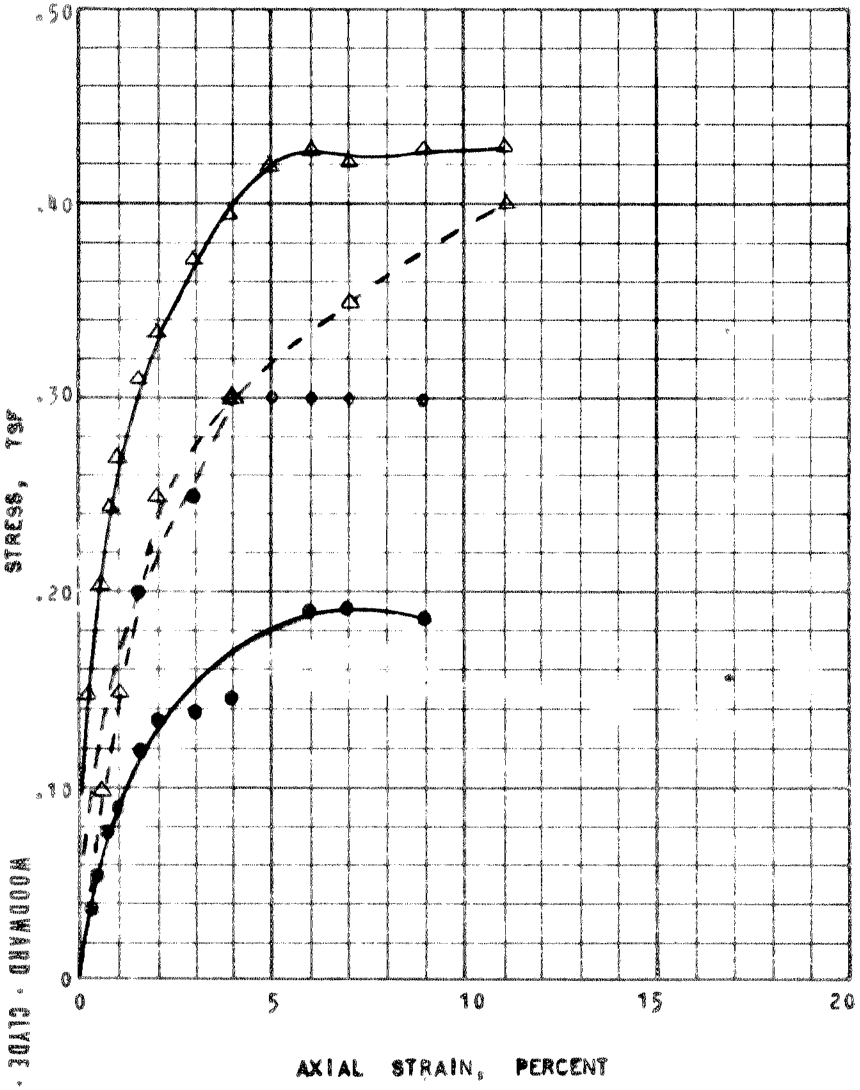
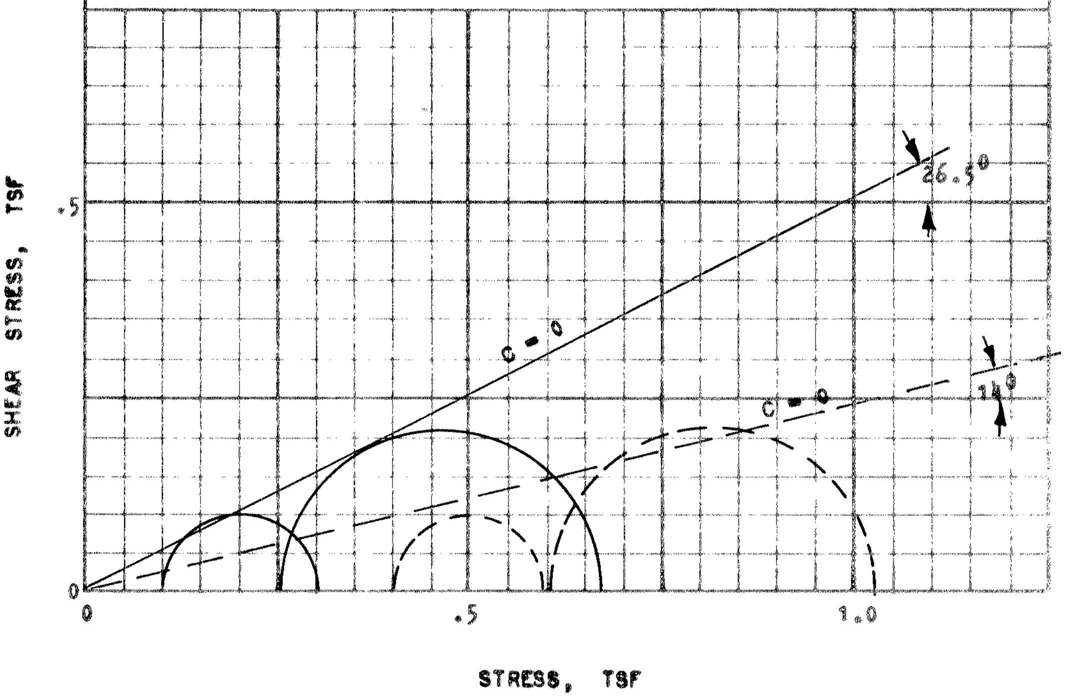
FIG. 58



DEVIATOR STRESS, $\sigma_1 - \sigma_3$ – SOLID LINES

PORE WATER PRESSURE, U – DASH LINES

SPECIMEN NO.	INITIAL MOISTURE CONTENT (%)	INITIAL DRY DENSITY (PCF)	CONFINING PRESSURE, (TSF)	DEVIATOR STRESS AT FAILURE (TSF)	PORE PRESSURE AT FAILURE (TSF)
△ 16-2-1B	90	50	.6	.42	.35
● 16-2-1A	90	50	.4	.19	.30



TYPE OF SPECIMEN : UNDISTURBED

TYPE OF TEST : C-U

EFFECTIVE STRESS – SOLID LINES

TOTAL STRESS – DASH LINES

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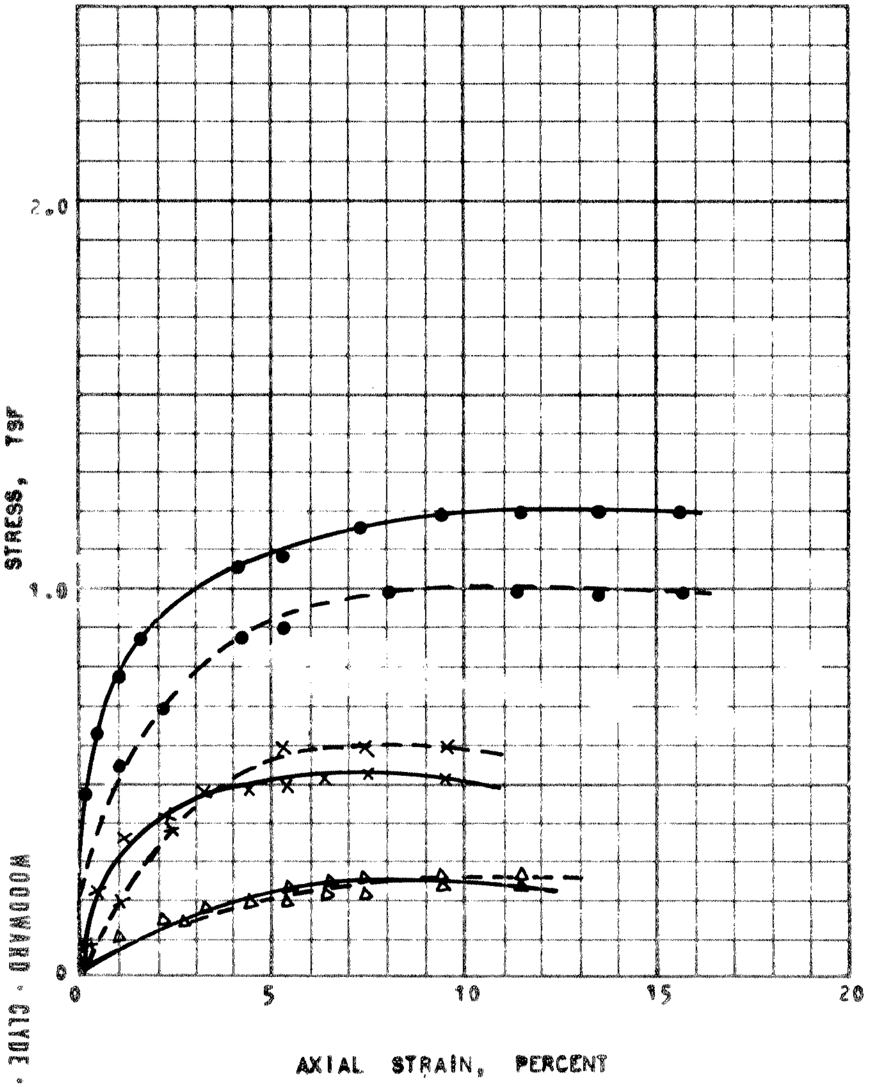
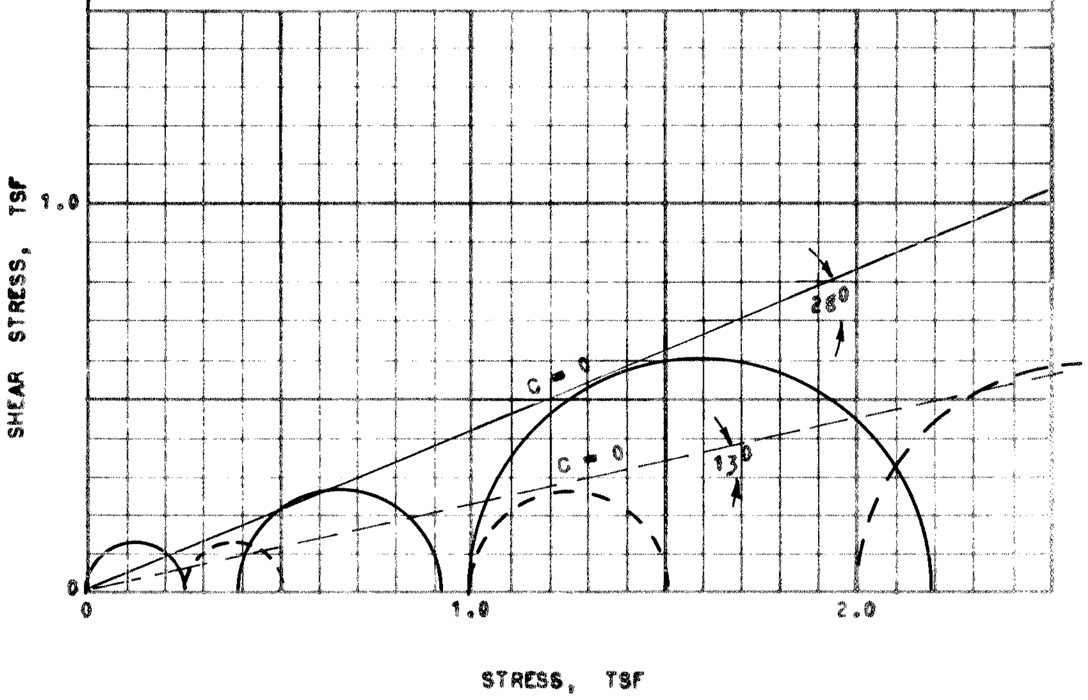
FIG. 10



DEVIATOR STRESS, $\sigma_1 - \sigma_3$ - SOLID LINES
PORE WATER PRESSURE, U - DASH LINES

SPECIMEN NO.	INITIAL MOISTURE CONTENT (%)	INITIAL DRY DENSITY (PCF)	CONFINING PRESSURE, (TSF)	DEVIATOR STRESS AT FAILURE (TSF)	PORE PRESSURE AT FAILURE (TSF)
—●— 19-2-3	86	51	2.00	1.20	1.00
—×— 19-2-2	94	47	1.00	0.53	0.60
—△— 19-2-4	77	55	0.25	0.25	0.25



TYPE OF SPECIMEN : UNDISTURBED
TYPE OF TEST : C-U

EFFECTIVE STRESS - SOLID LINES
TOTAL STRESS - DASH LINES

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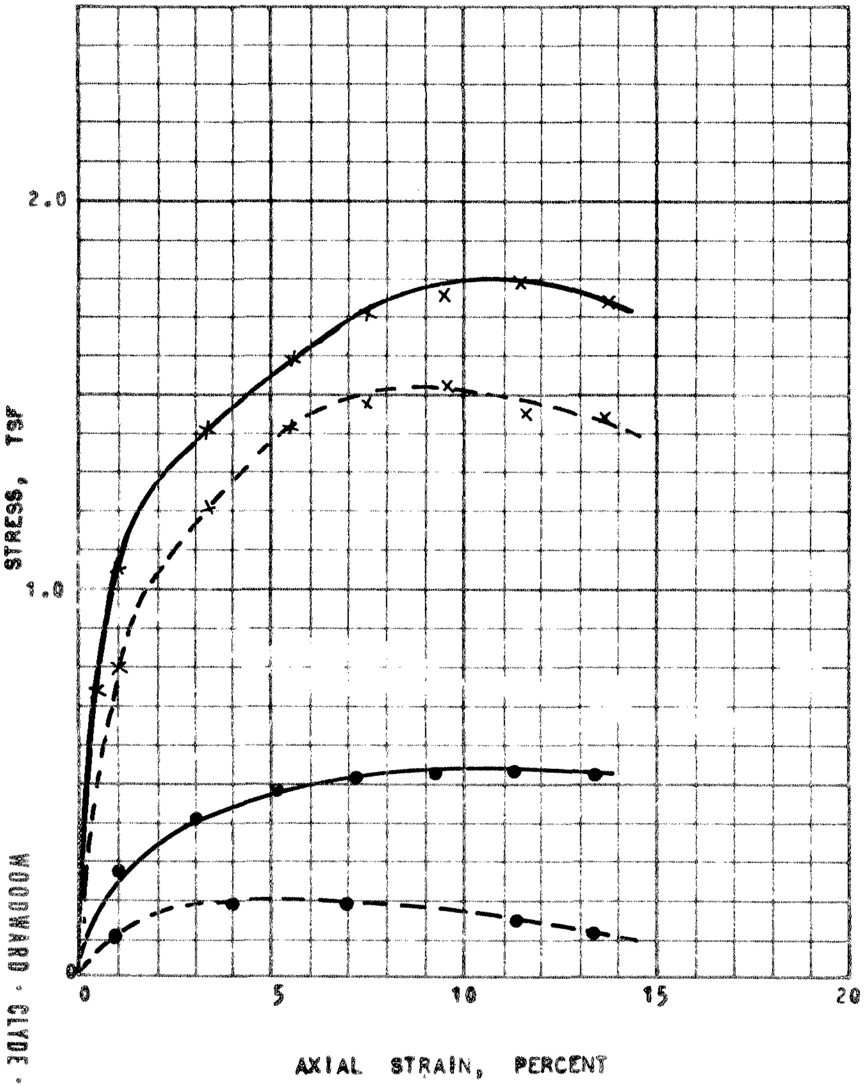
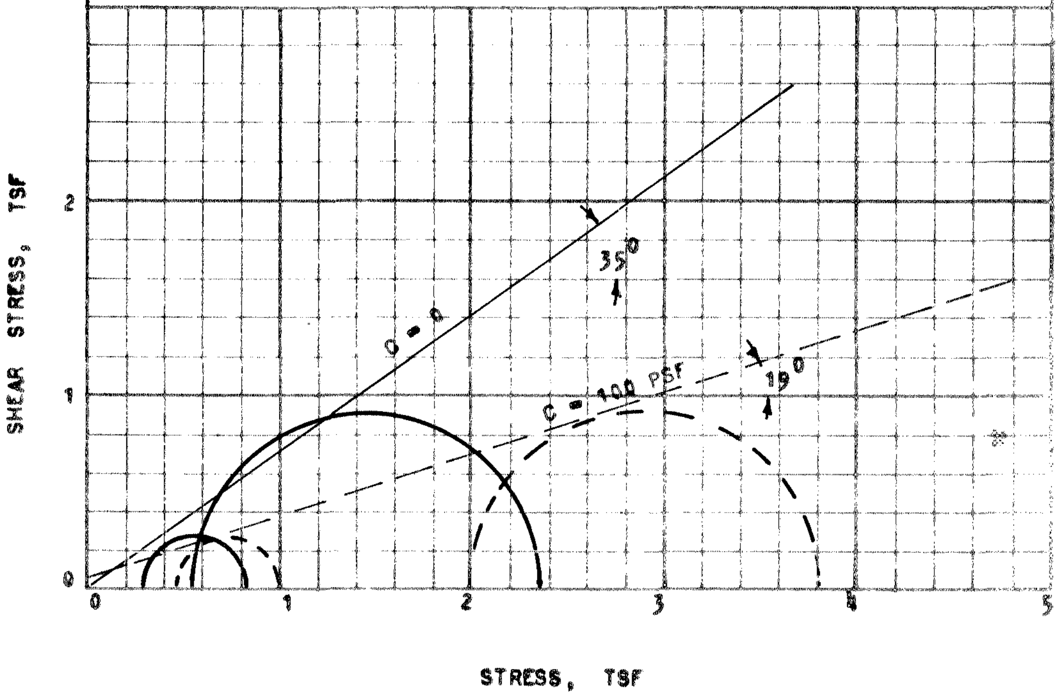
FIG. 60



DEVIATOR STRESS, $\sigma_1 - \sigma_3$ – SOLID LINES

PORE WATER PRESSURE, U – DASH LINES

SPECIMEN NO.	INITIAL MOISTURE CONTENT (%)	INITIAL DRY DENSITY (PCF)	CONFINING PRESSURE, (TSF)	DEVIATOR STRESS AT FAILURE (TSF)	PORE PRESSURE AT FAILURE (TSF)
—●— 22-1-3	62	61	0.50	0.55	0.20
—×— 22-1-2	45	71	2.00	1.80	1.45



TYPE OF SPECIMEN : UNDISTURBED

TYPE OF TEST : C-U

EFFECTIVE STRESS – SOLID LINES

TOTAL STRESS – DASH LINES

TRIAXIAL COMPRESSION TEST

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FIG. 68

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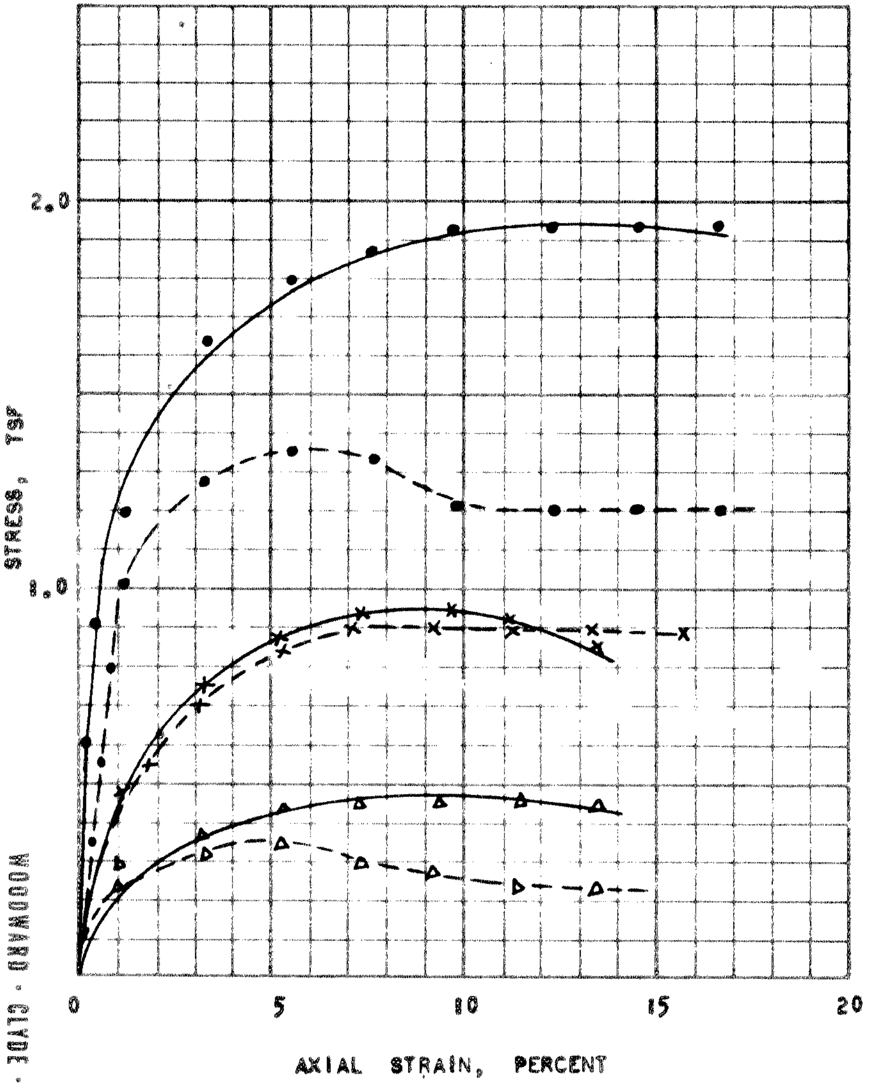
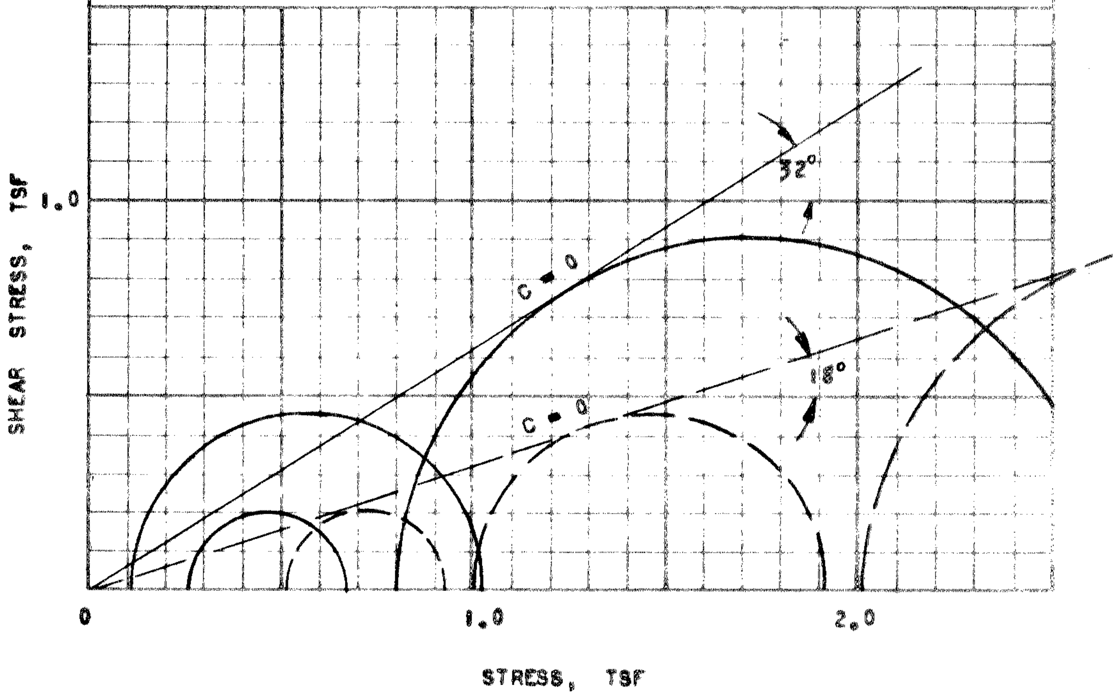


SPECIMEN NO.	INITIAL MOISTURE CONTENT (%)	INITIAL DRY DENSITY (PCF)	CONFINING PRESSURE, (TSF)	DEVIATOR STRESS AT FAILURE (TSF)	PORE PRESSURE AT FAILURE (TSF)
—●— 31-3-4	34	87	2.00	1.93	1.20
—×— 31-3-2	66	57	1.00	0.93	0.90
—△— 31-3-3	37	83	0.50	0.45	0.25



DEVIATOR STRESS, $\sigma_1 - \sigma_3$ – SOLID LINES

PORE WATER PRESSURE, U – DASH LINES

TYPE OF SPECIMEN : UNDISTURBED

TYPE OF TEST : CONSOLIDATED UNDRAINED

EFFECTIVE STRESS – SOLID LINES

TOTAL STRESS – DASH LINES

TRIAXIAL COMPRESSION TEST

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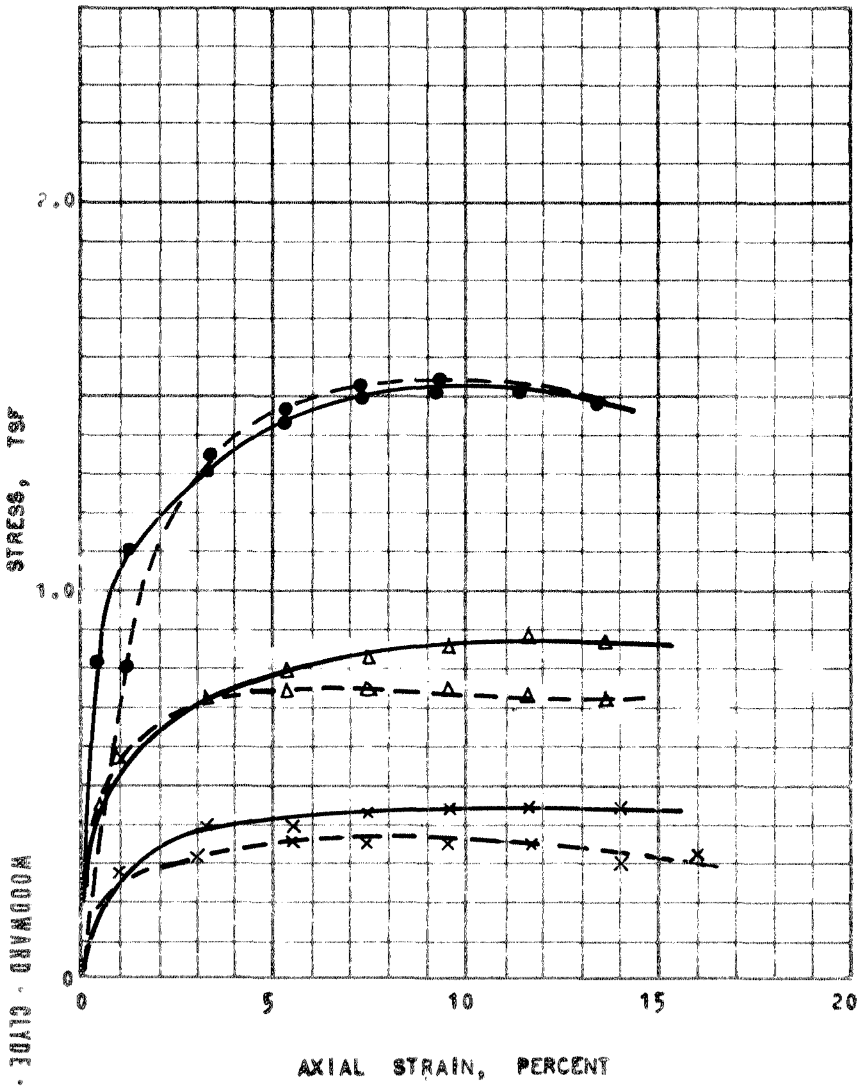
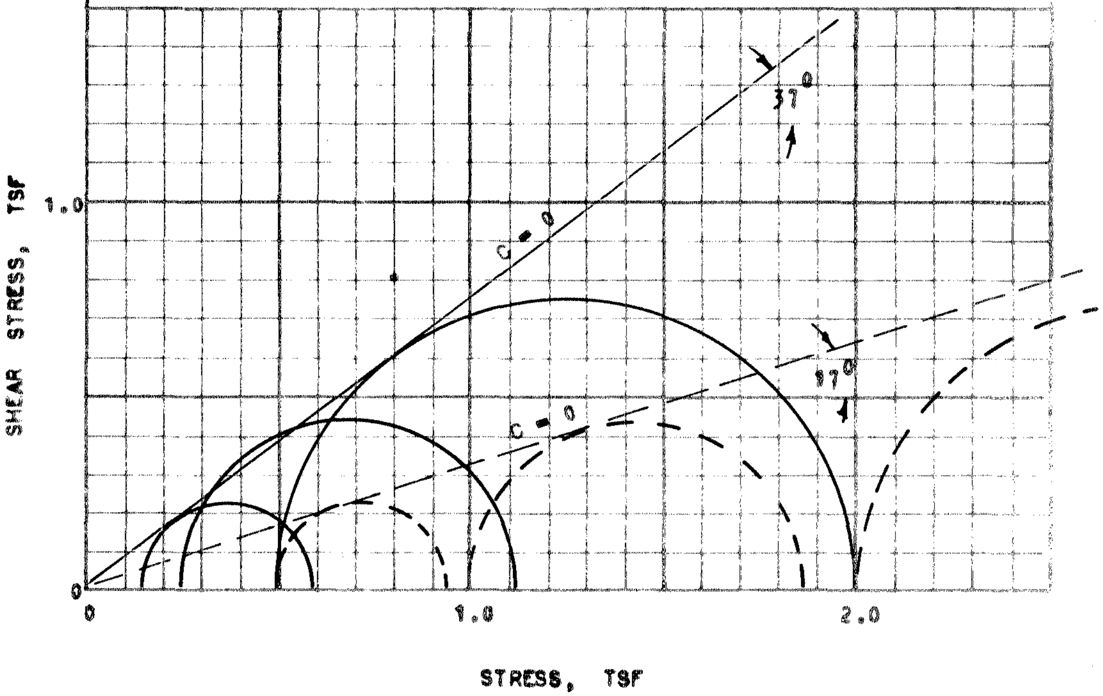
FIG. 62



DEVIATOR STRESS, $\sigma_1 - \sigma_3$ – SOLID LINES

PORE WATER PRESSURE, U – DASH LINES

SPECIMEN NO.	INITIAL MOISTURE CONTENT (%)	INITIAL DRY DENSITY (PCF)	CONFINING PRESSURE, (TSF)	DEVIATOR STRESS AT FAILURE (TSF)	PORE PRESSURE AT FAILURE (TSF)
—●—35-3-3	51	70	2.0	1.50	1.50
—△—35-3-2	56	67	1.0	0.85	0.75
—×—35-3-1	88	49	0.5	0.45	0.35



TYPE OF SPECIMEN : UNDISTURBED

TYPE OF TEST : C–U

EFFECTIVE STRESS – SOLID LINES

TOTAL STRESS – DASH LINES

TRIAXIAL COMPRESSION TEST

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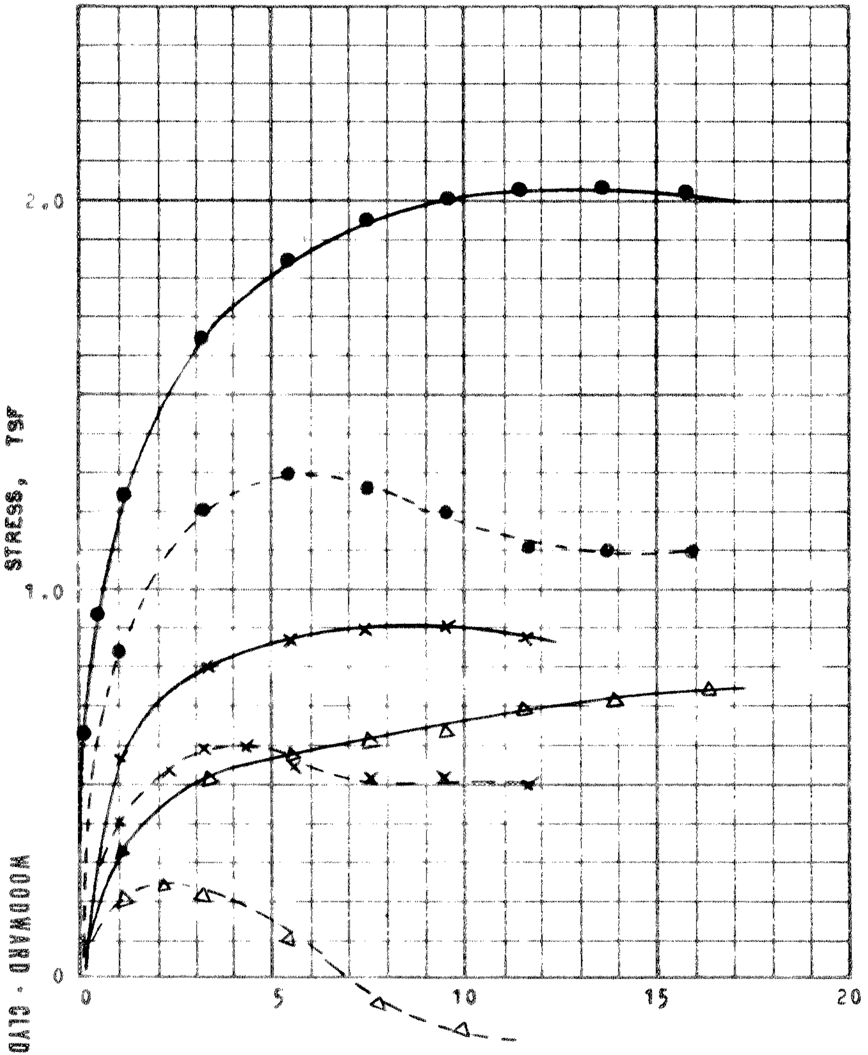
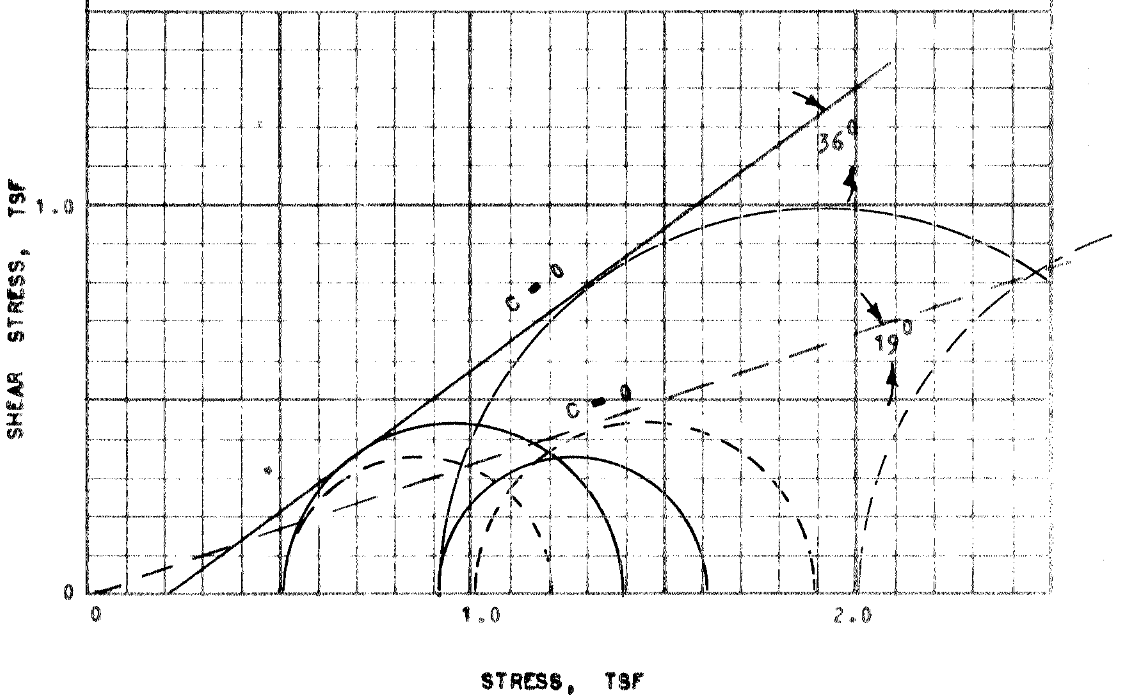
FIG. 63



SPECIMEN NO.	INITIAL MOISTURE CONTENT (%)	INITIAL DRY DENSITY (PCF)	CONFINING PRESSURE, (TSF)	DEVIATOR STRESS AT FAILURE (TSF)	PORE PRESSURE AT FAILURE (TSF)
● 38-3-4	34	88	2.00	2.04	+1.10
× 38-3-2	43	79	1.00	0.90	+0.50
△ 38-3-3	38	83	0.50	0.70	-0.40



DEVIATOR STRESS, $\sigma_1 - \sigma_3$ – SOLID LINES

PORE WATER PRESSURE, U – DASH LINES

TYPE OF SPECIMEN : UNDISTURBED

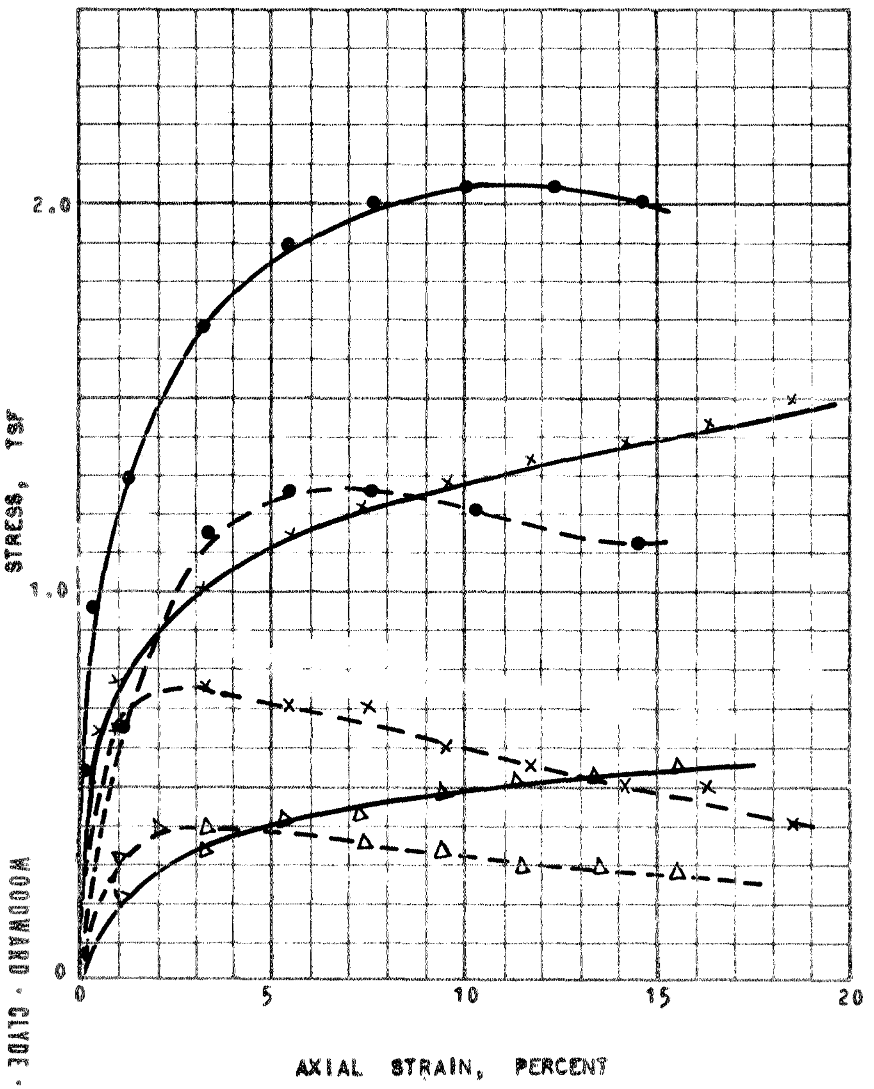
TYPE OF TEST : C – U

EFFECTIVE STRESS – SOLID LINES

TOTAL STRESS – DASH LINES

TRIAXIAL COMPRESSION TEST

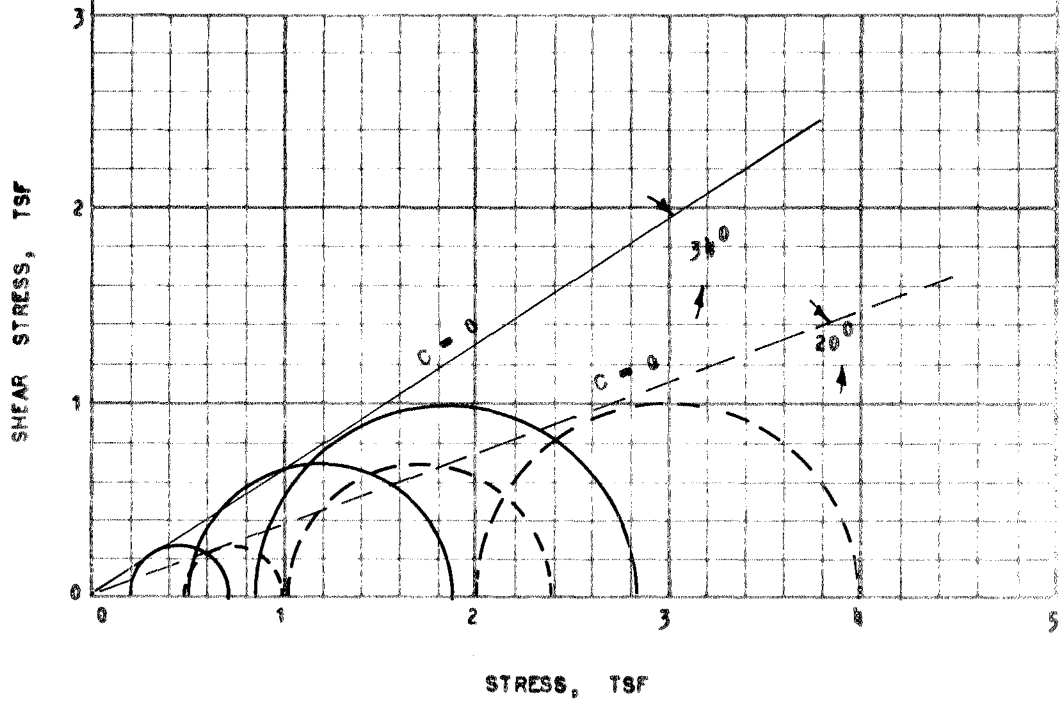
FIG. 68



DEVIATOR STRESS, $\sigma_1 - \sigma_3$ – SOLID LINES

PORE WATER PRESSURE, U – DASH LINES

SPECIMEN NO.	INITIAL MOISTURE CONTENT (%)	INITIAL DRY DENSITY (PCF)	CONFINING PRESSURE, (TSF)	DEVIATOR STRESS AT FAILURE (TSF)	PORE PRESSURE AT FAILURE (TSF)
—●— B3-2-2	26	97	1.95	2.00	1.10
—×— B3-2-4	30	95	1.00	1.40	0.50
—△— B3-2-3	25	101	0.50	0.55	0.30



TYPE OF SPECIMEN : UNDISTURBED

TYPE OF TEST : C-U

EFFECTIVE STRESS – SOLID LINES

TOTAL STRESS – DASH LINES

TRIAXIAL COMPRESSION TEST

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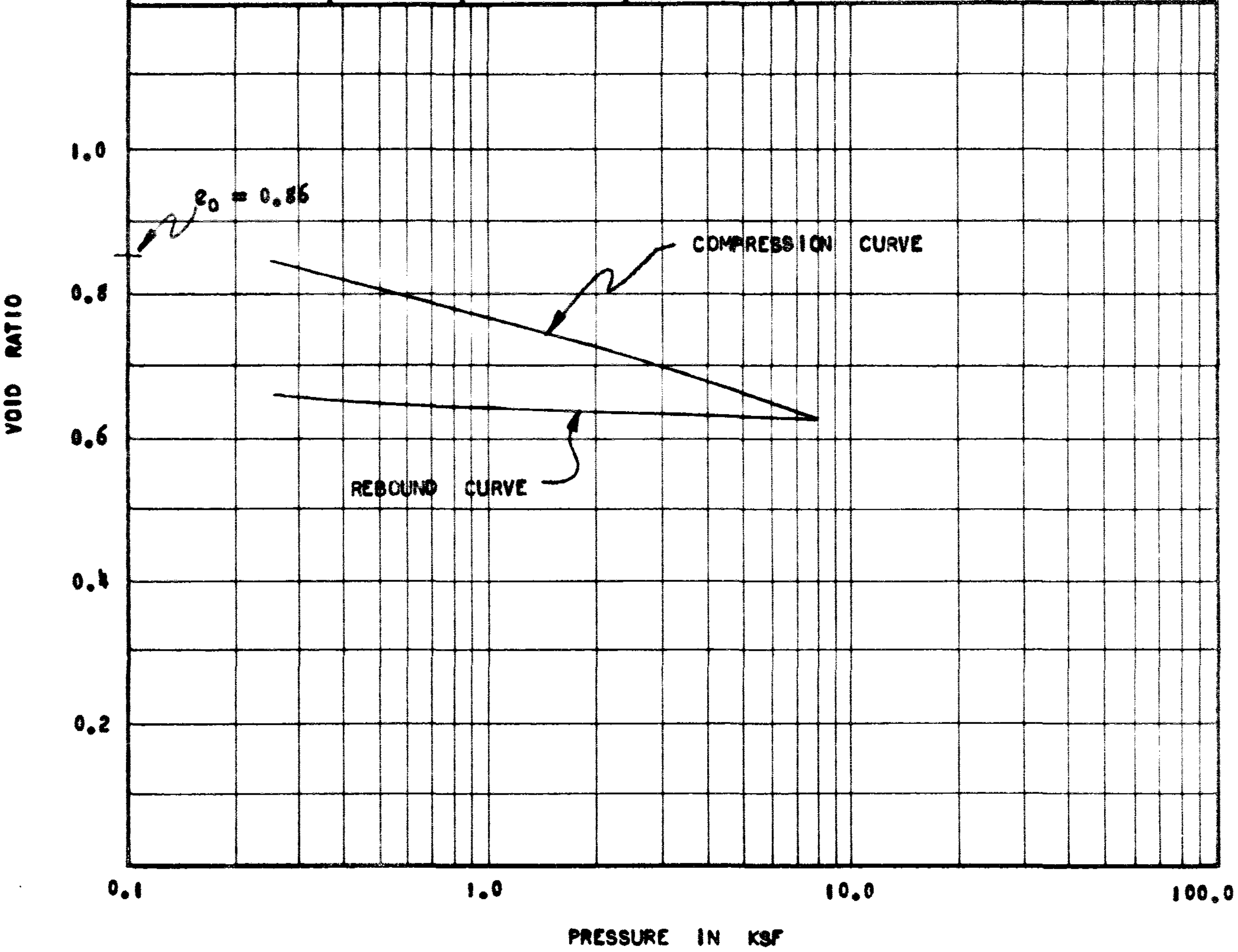
FIG. 63

SAMPLE NO. 7-3-4	SUMMARY OF TEST RESULTS					
	SPECIFIC GRAVITY	MOISTURE CONTENT, (%)	DRY DENSITY (PCF)	PER CENT OF SATURATION, (%)	HEIGHT (IN.)	DIAMETER (IN.)
INITIAL	2.62	31.8	87.8	96.6	0.931	1.94
FINAL	(ASSUMED)	25.7	96.9	98.1	0.844	



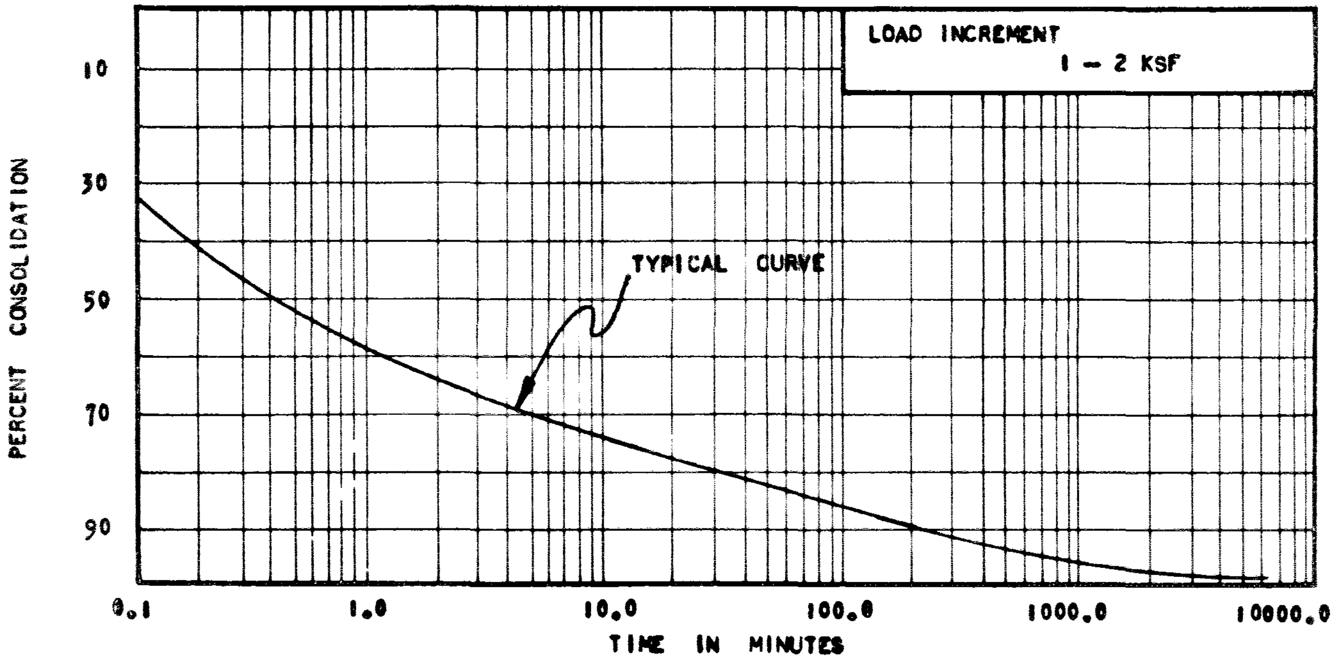


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FIG. 66

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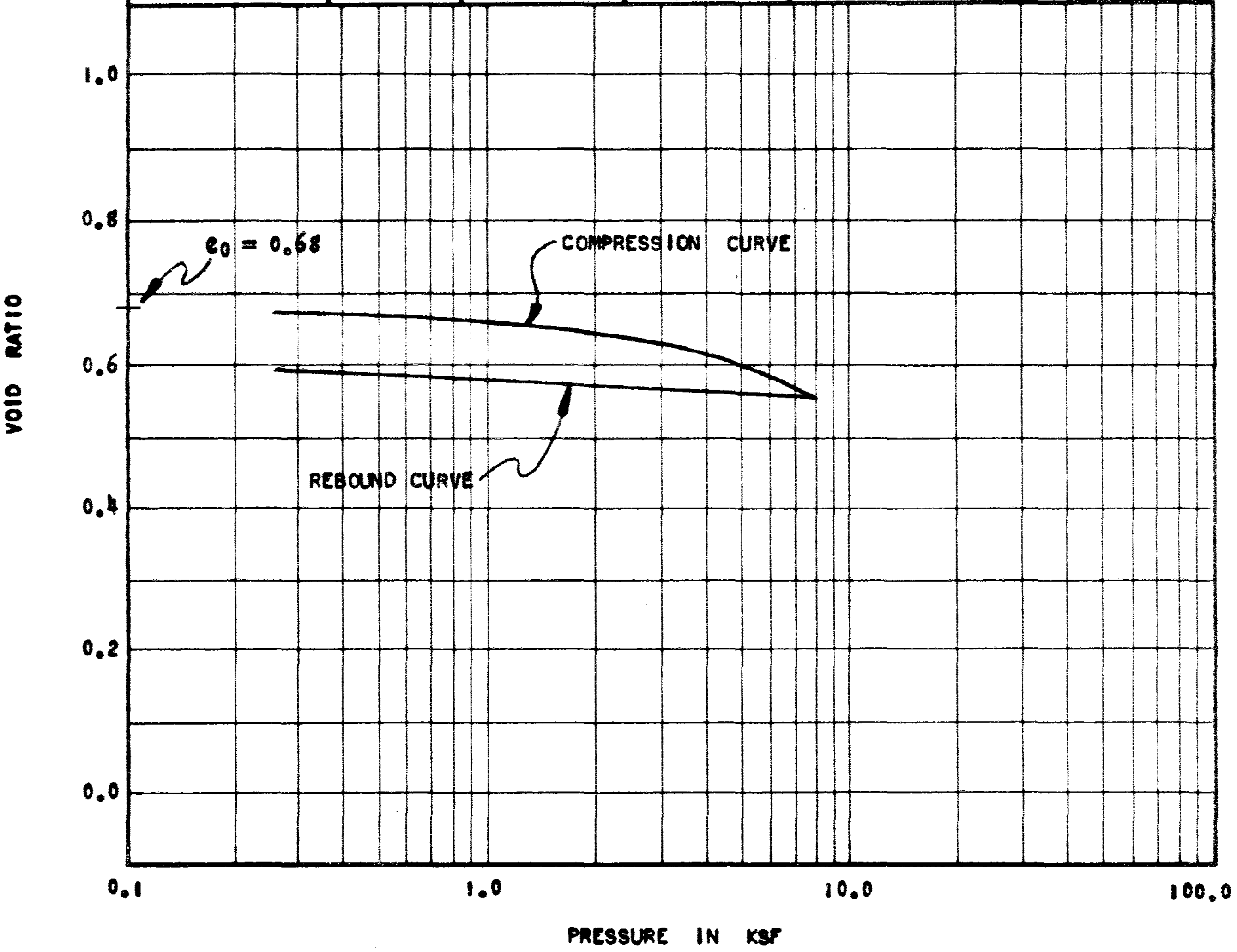
SAMPLE NO. 14-3-4	SUMMARY OF TEST RESULTS					
	SPECIFIC GRAVITY	MOISTURE CONTENT, (%)	DRY DENSITY (PCF)	PER CENT OF SATURATION, (%)	HEIGHT (IN.)	DIAMETER (IN.)
INITIAL	2.71	24.2	100.8	96.7	0.946	1.94
FINAL	(ASSUMED)	23.1	105.3	103.1	0.906	



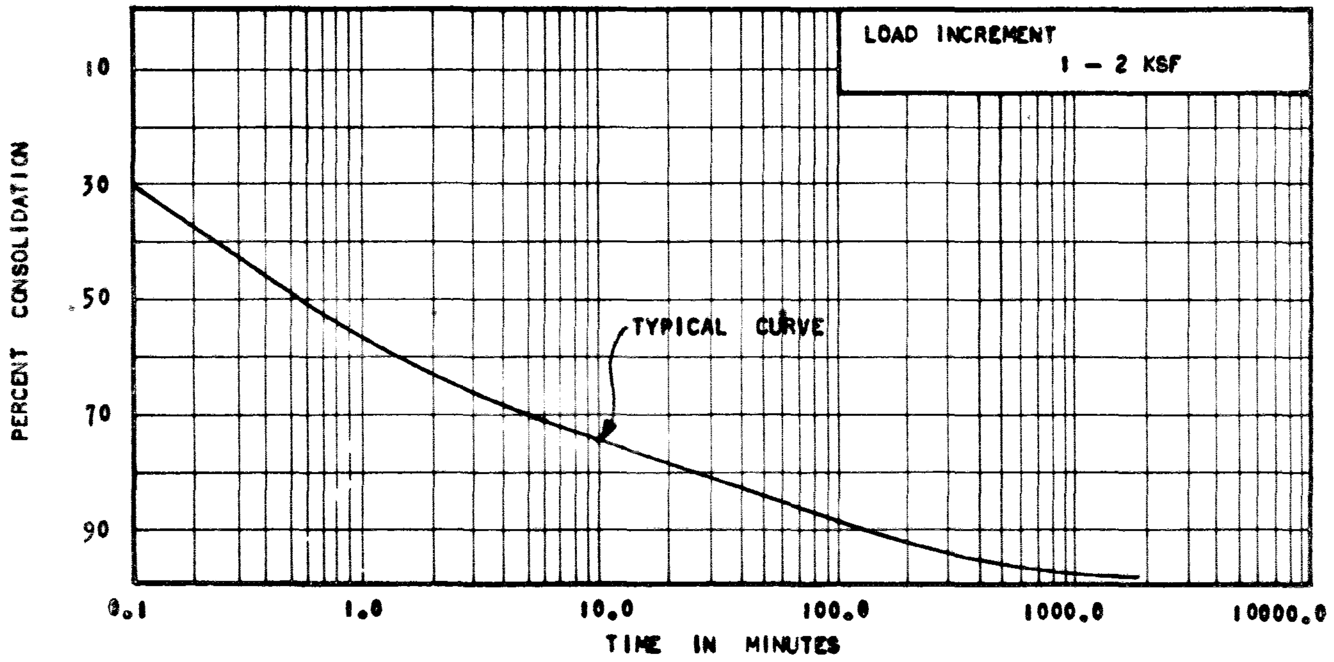


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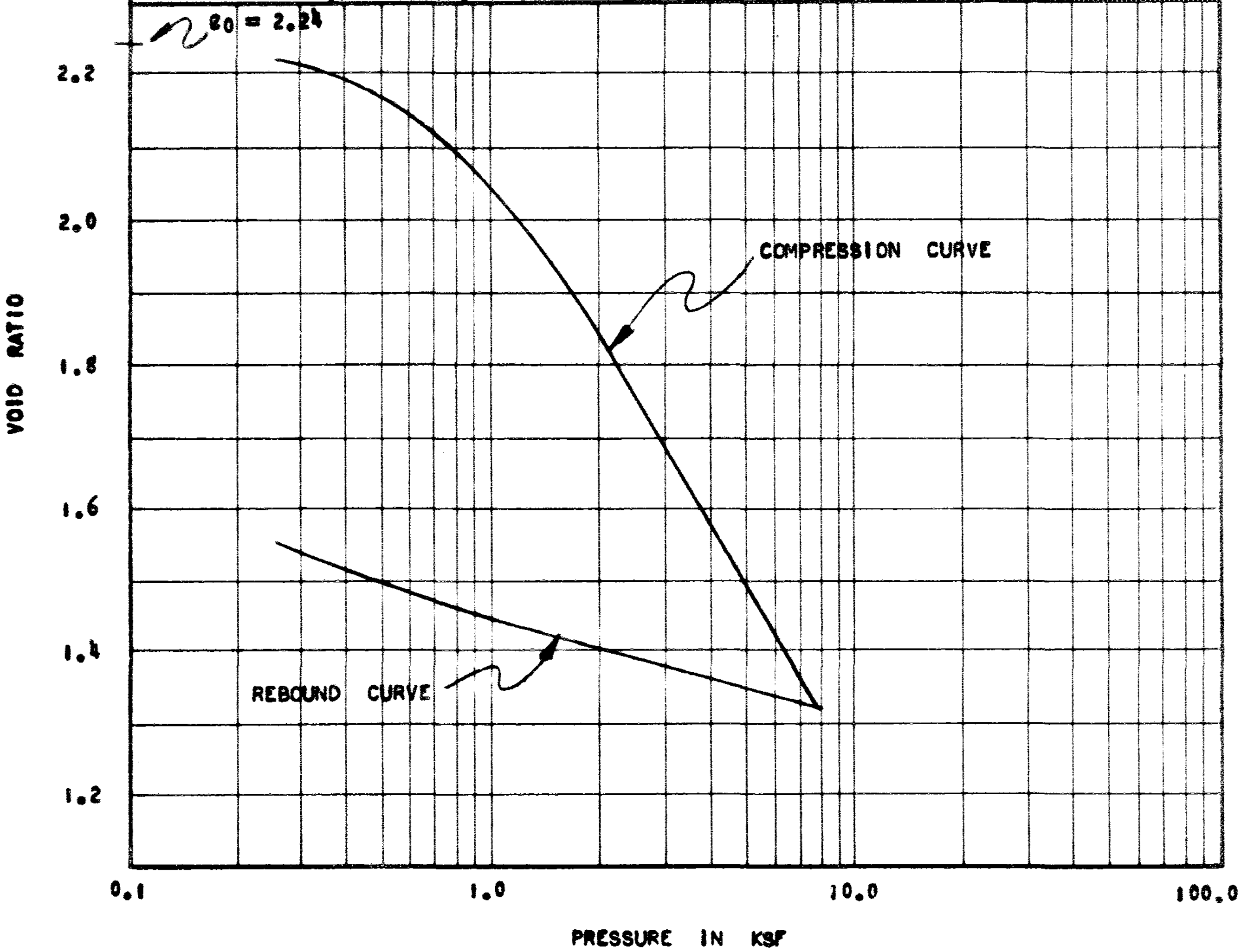
FIG. (

SAMPLE NO. 19-1-4	SUMMARY OF TEST RESULTS					
	SPECIFIC GRAVITY	MOISTURE CONTENT, (%)	DRY DENSITY (PCF)	PER CENT OF SATURATION, (%)	HEIGHT (IN.)	DIAMETER (IN.)
INITIAL	2.69	81.9	51.8	98.5	0.939	1.94
FINAL	(ASSUMED)	60.0	64.5	100.8	0.754	



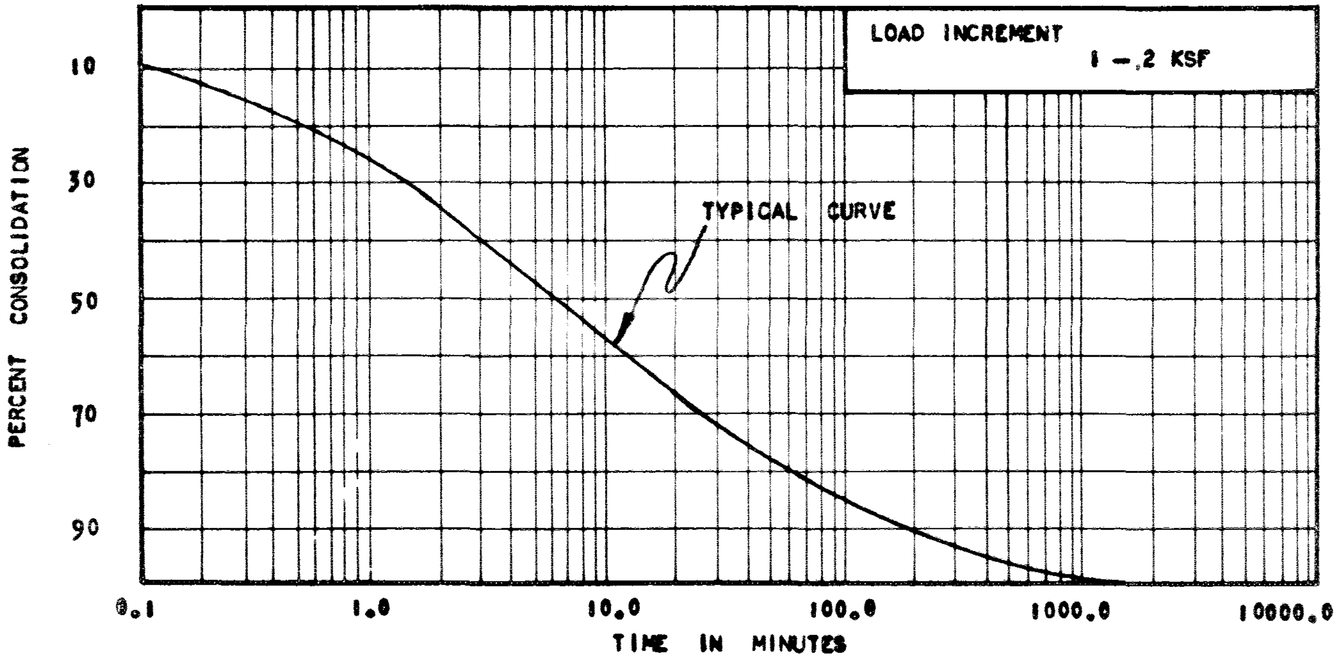


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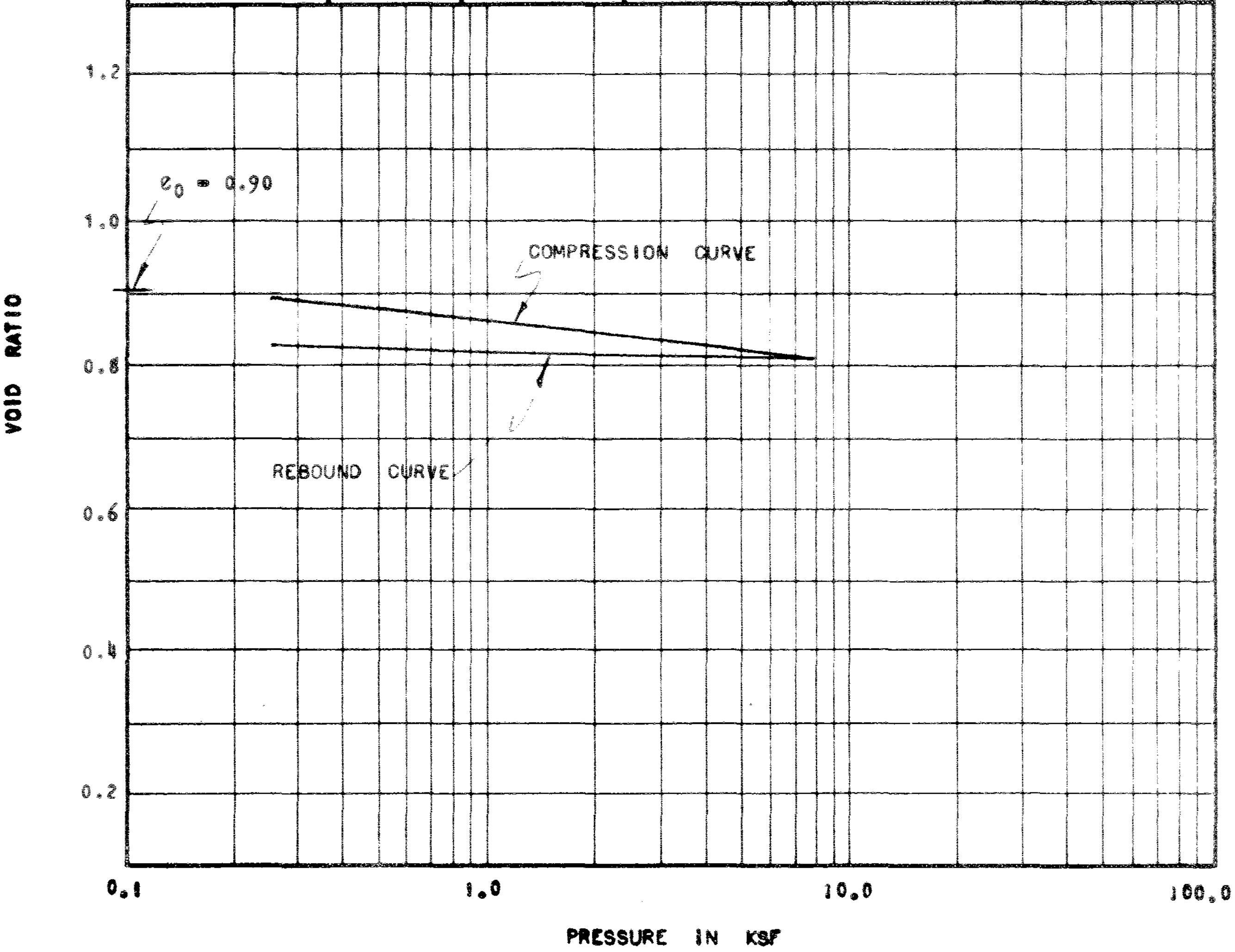
FIG. 68

SAMPLE NO. 28-1	SUMMARY OF TEST RESULTS					
	SPECIFIC GRAVITY	MOISTURE CONTENT, (%)	DRY DENSITY (PCF)	PER CENT OF SATURATION, (%)	HEIGHT (IN.)	DIAMETER (IN.)
INITIAL	ASSUMED 2.67	33.7	87.4	99.5	0.922	2.80
FINAL		31.6	90.4	100.0	0.892	



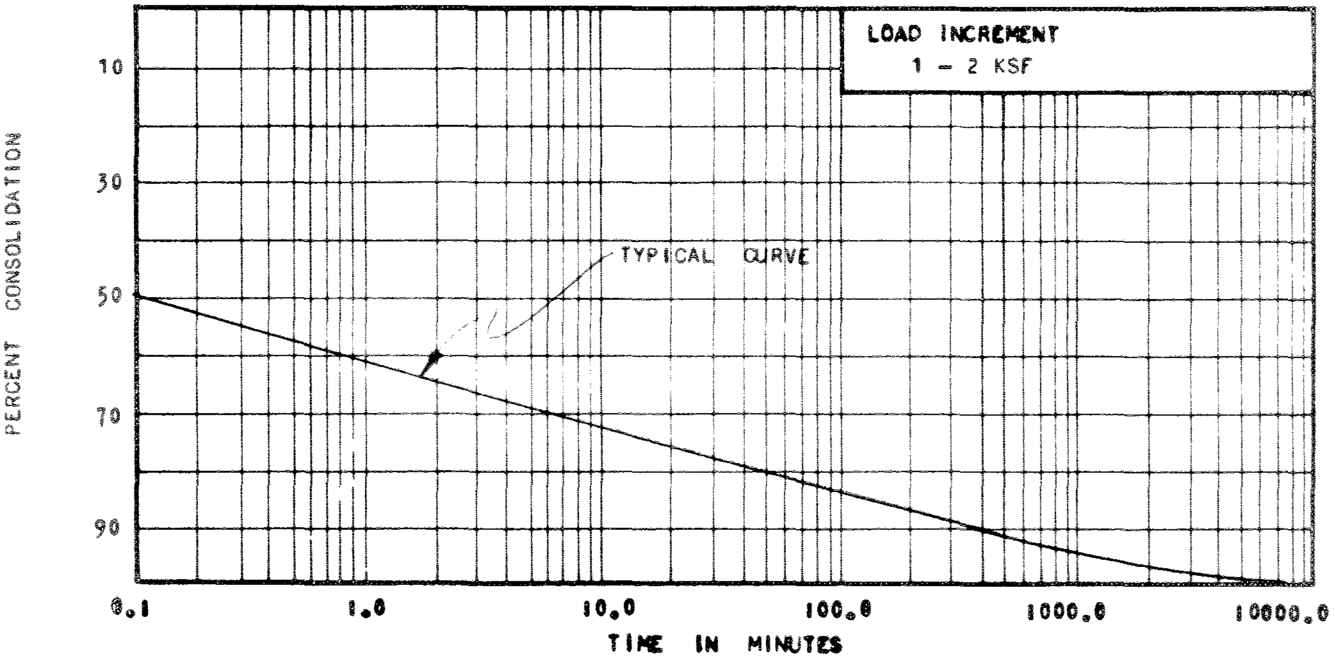


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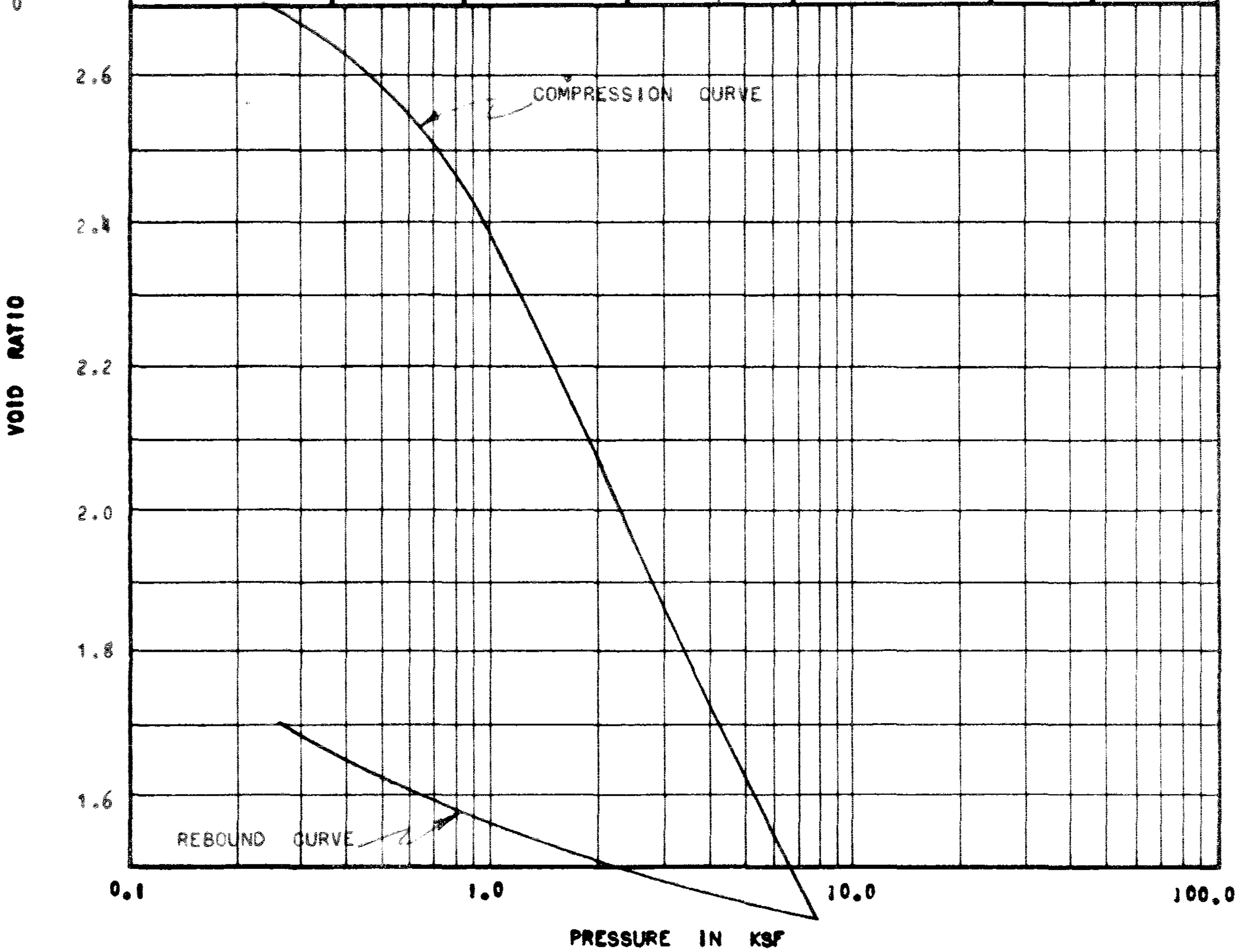
FIG. 69

SAMPLE NO. 29-4	SUMMARY OF TEST RESULTS					
	SPECIFIC GRAVITY	MOISTURE CONTENT, (%)	DRY DENSITY (PCF)	PER CENT OF SATURATION, (%)	HEIGHT (IN.)	DIAMETER (IN.)
INITIAL	ASSUMED 2.66	100.8	43.9	96.4	0.923	2.80
FINAL		64.6	60.7	99.1	0.668	

$e_0 = 2.78$



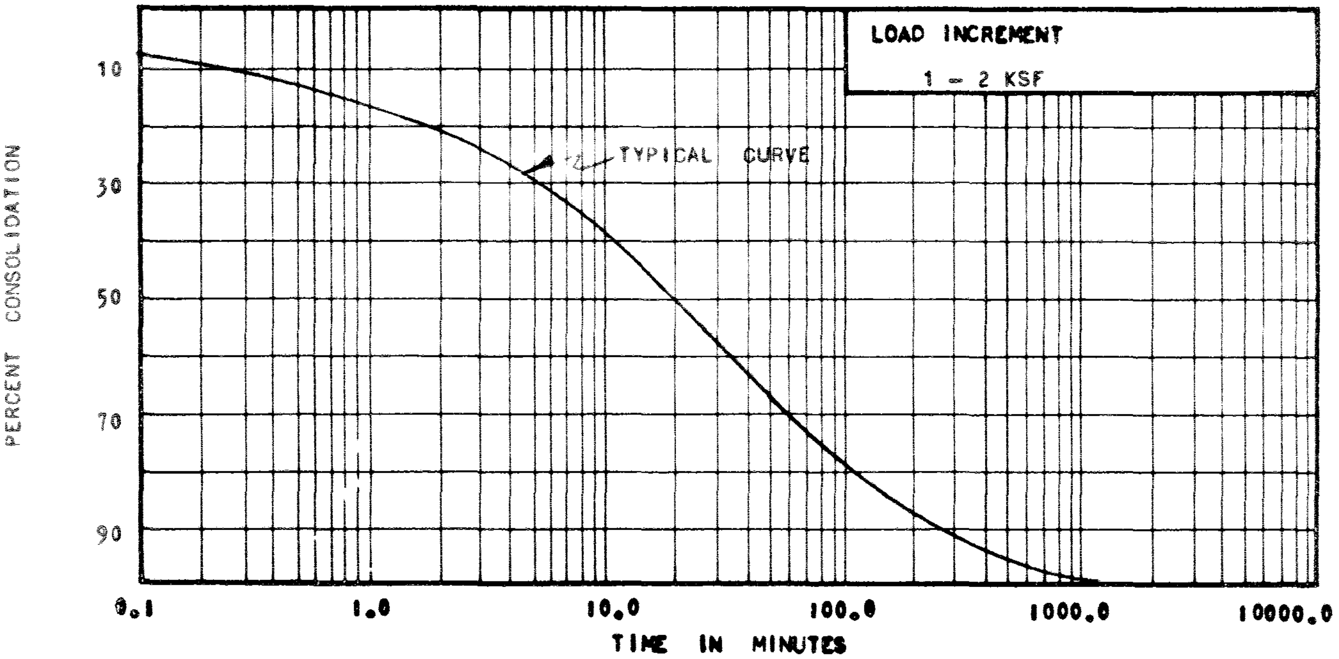


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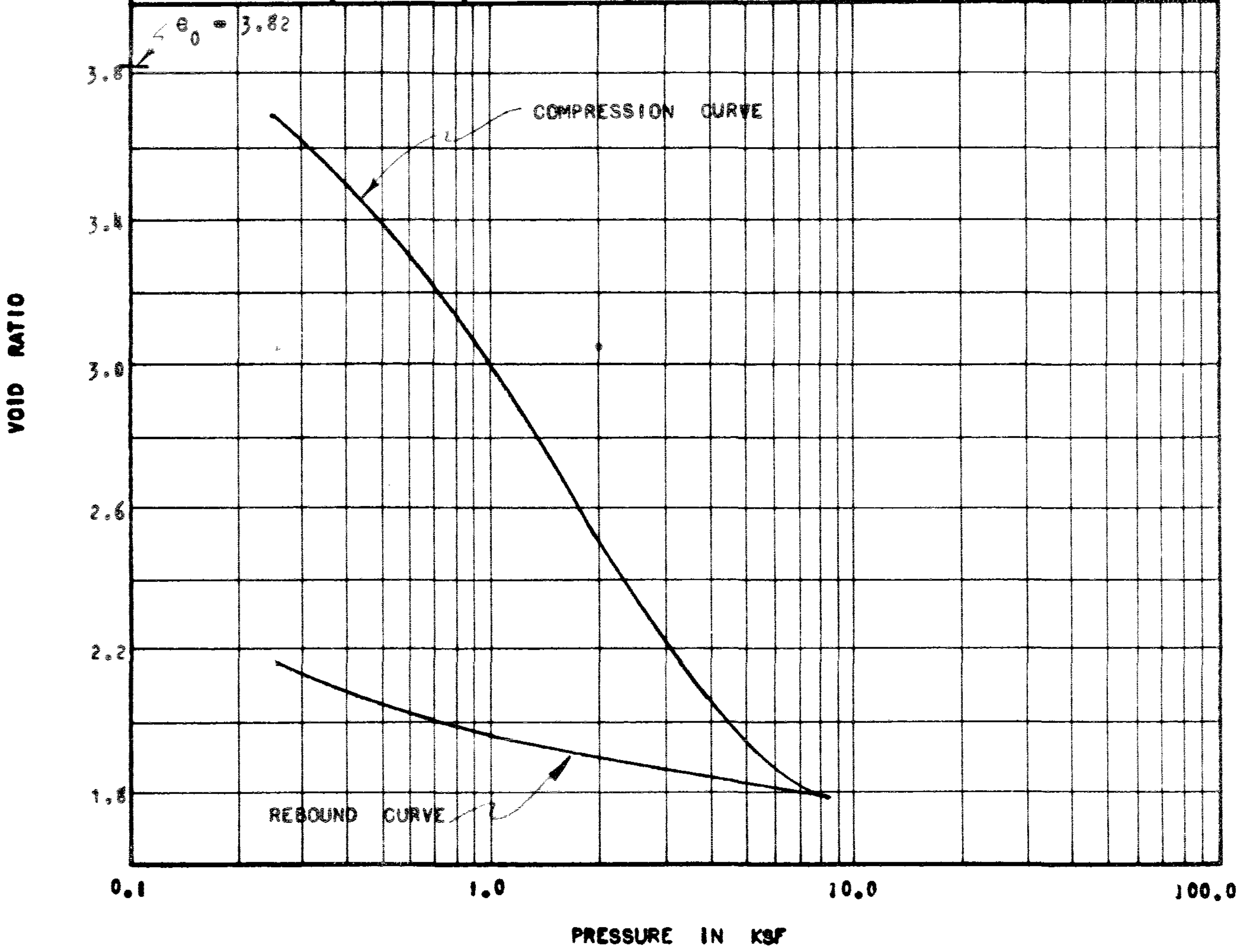
FIG. 70

SAMPLE NO. 31-2-2	SUMMARY OF TEST RESULTS					
	SPECIFIC GRAVITY	MOISTURE CONTENT, (%)	DRY DENSITY (PCF)	PER CENT OF SATURATION, (%)	HEIGHT (IN.)	DIAMETER (IN.)
INITIAL	2.70	134.1	35.0	94.8	0.939	1.94
FINAL	(ASSUMED)	81.8	52.0	98.4	0.632	



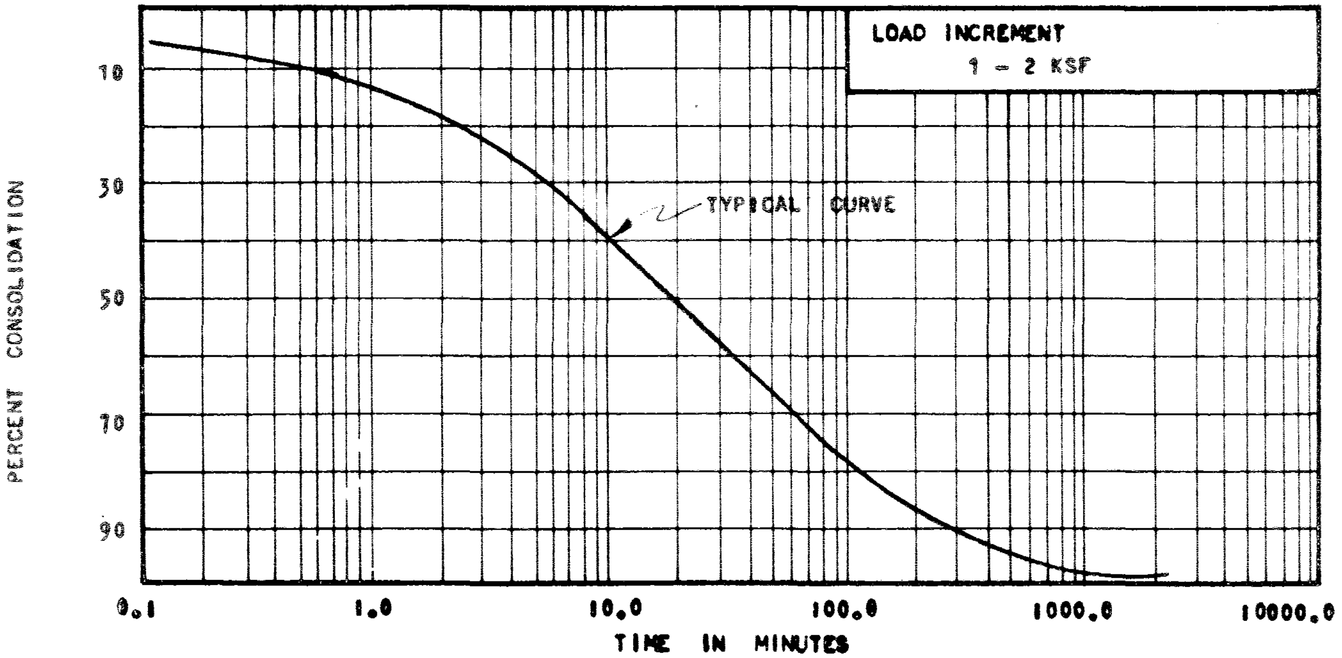


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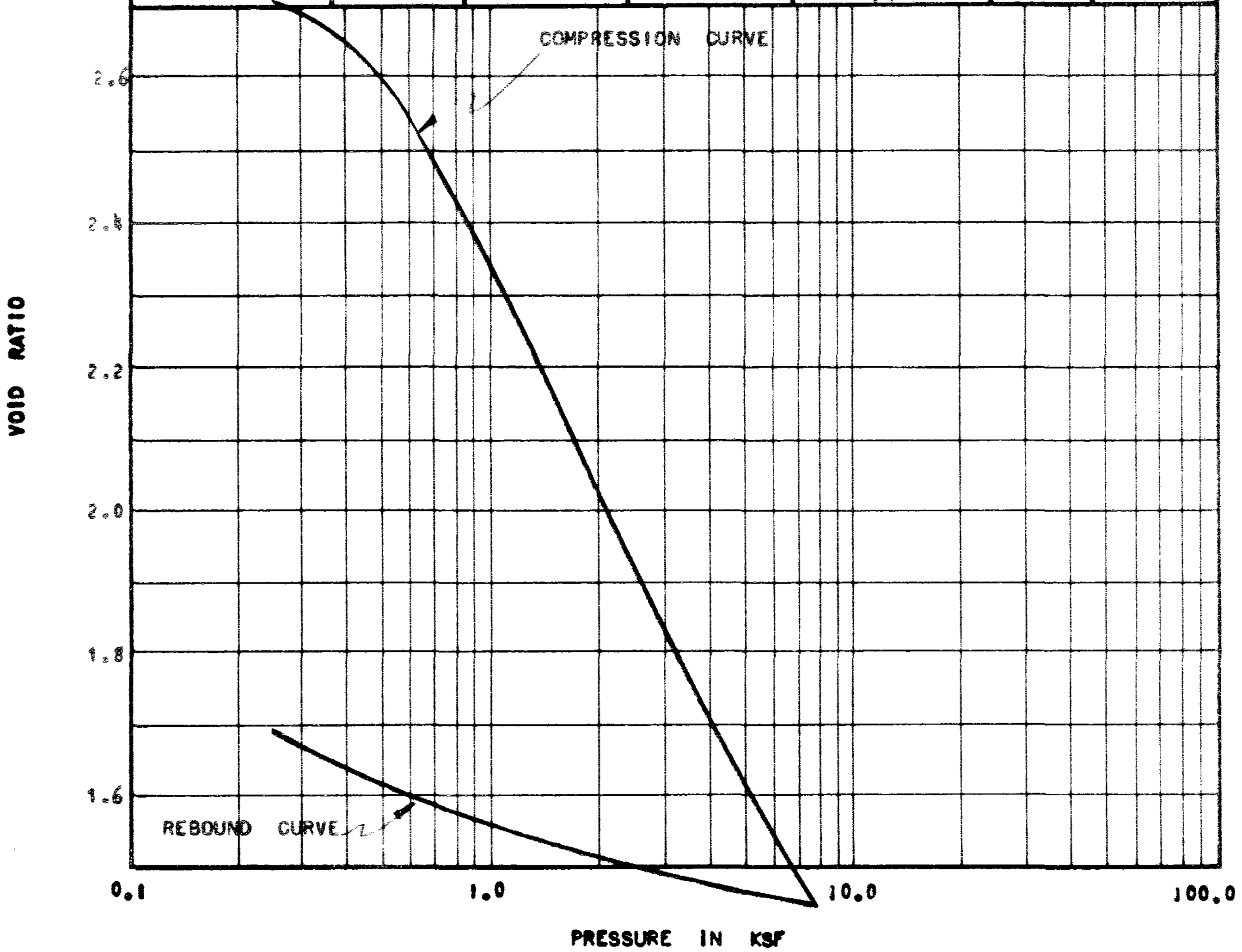
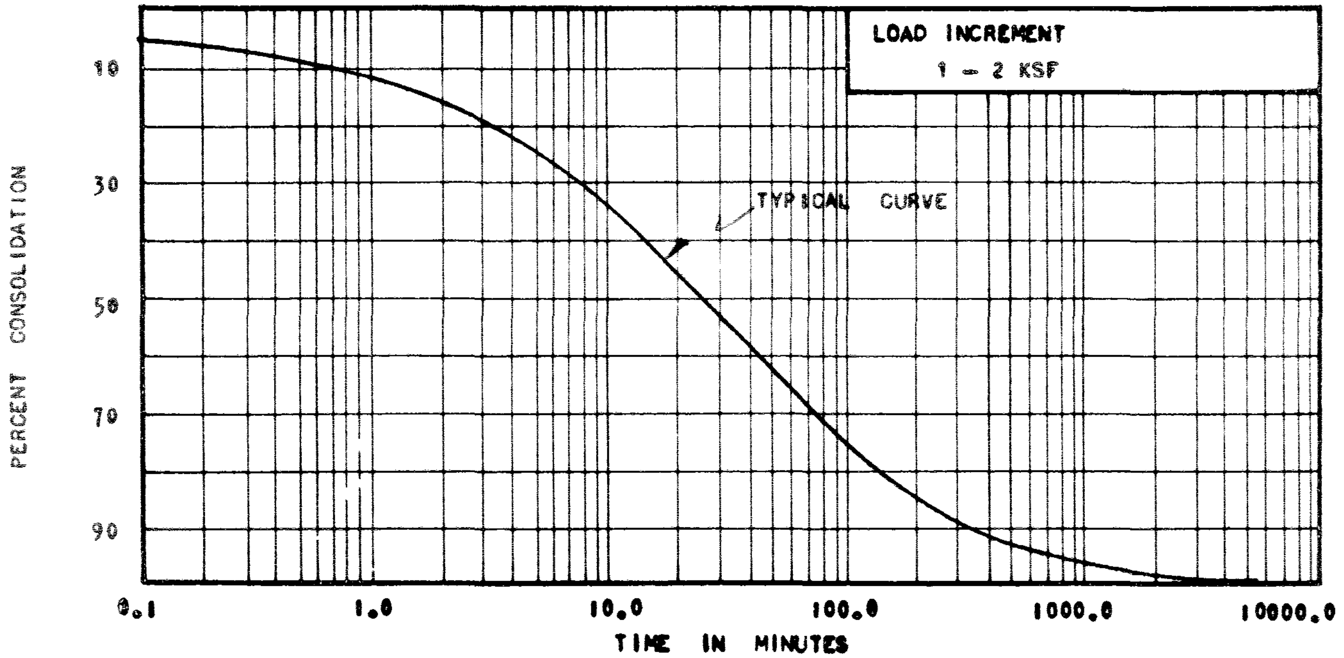
FIG. 71

$e_0 = 2.74$

SAMPLE NO. 35-2-4	SUMMARY OF TEST RESULTS					
	SPECIFIC GRAVITY	MOISTURE CONTENT, (%)	DRY DENSITY (PCF)	PER CENT OF SATURATION, (%)	HEIGHT (IN.)	DIAMETER (IN.)
INITIAL	2.65 (ASSUMED)	100.7	44.2	97.4	0.946	1.94
FINAL		64.6	60.1	97.6	0.696	





CONSOLIDATION TEST

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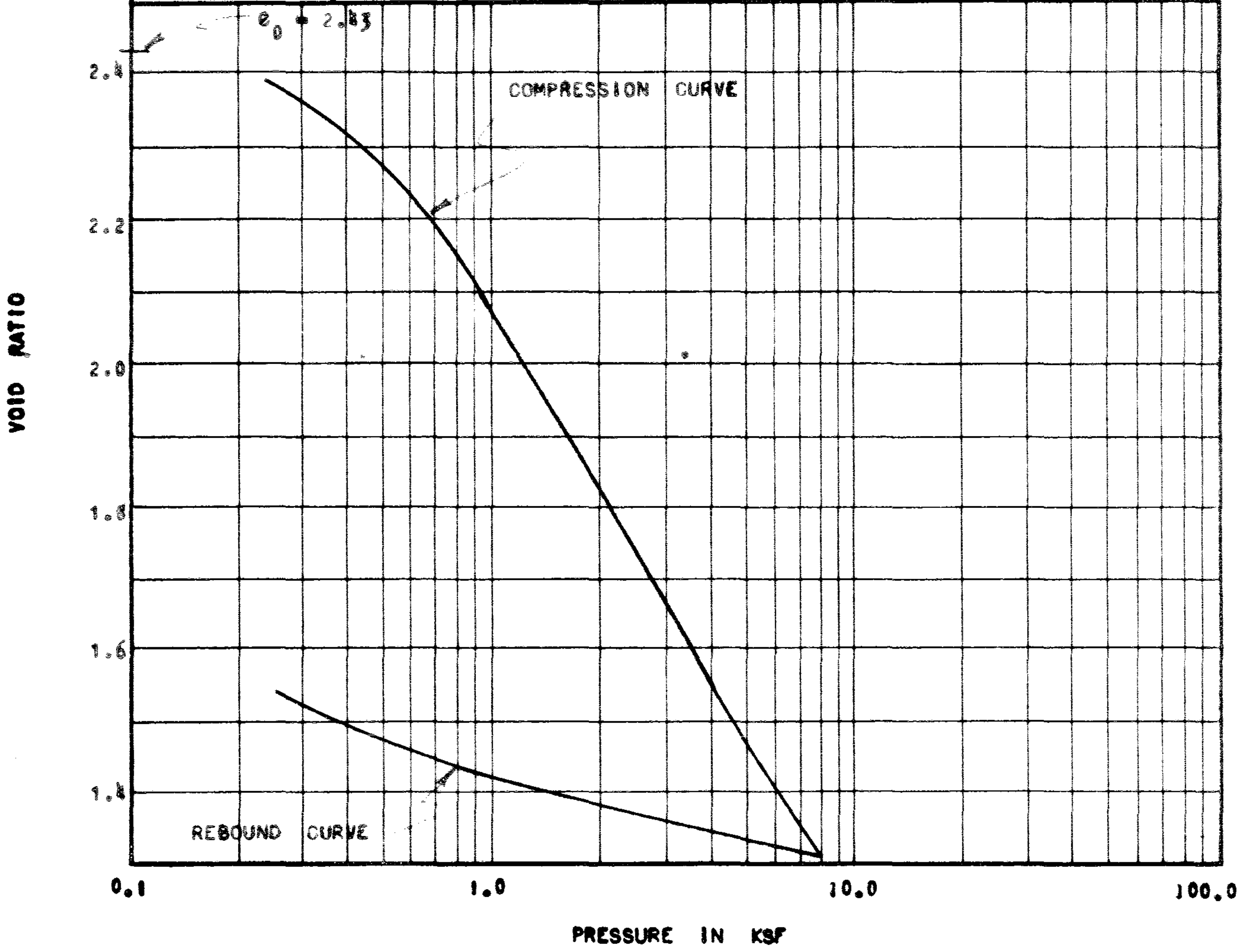
FIG. 23

SAMPLE NO. 41-4-4	SUMMARY OF TEST RESULTS					
	SPECIFIC GRAVITY	MOISTURE CONTENT, (%)	DRY DENSITY (PCF)	PER CENT OF SATURATION, (%)	HEIGHT (IN.)	DIAMETER (IN.)
INITIAL	2.69 (ASSUMED)	90.2	49.0	100.0	0.935	1.94
FINAL		59.5	64.6	100.0	0.709	



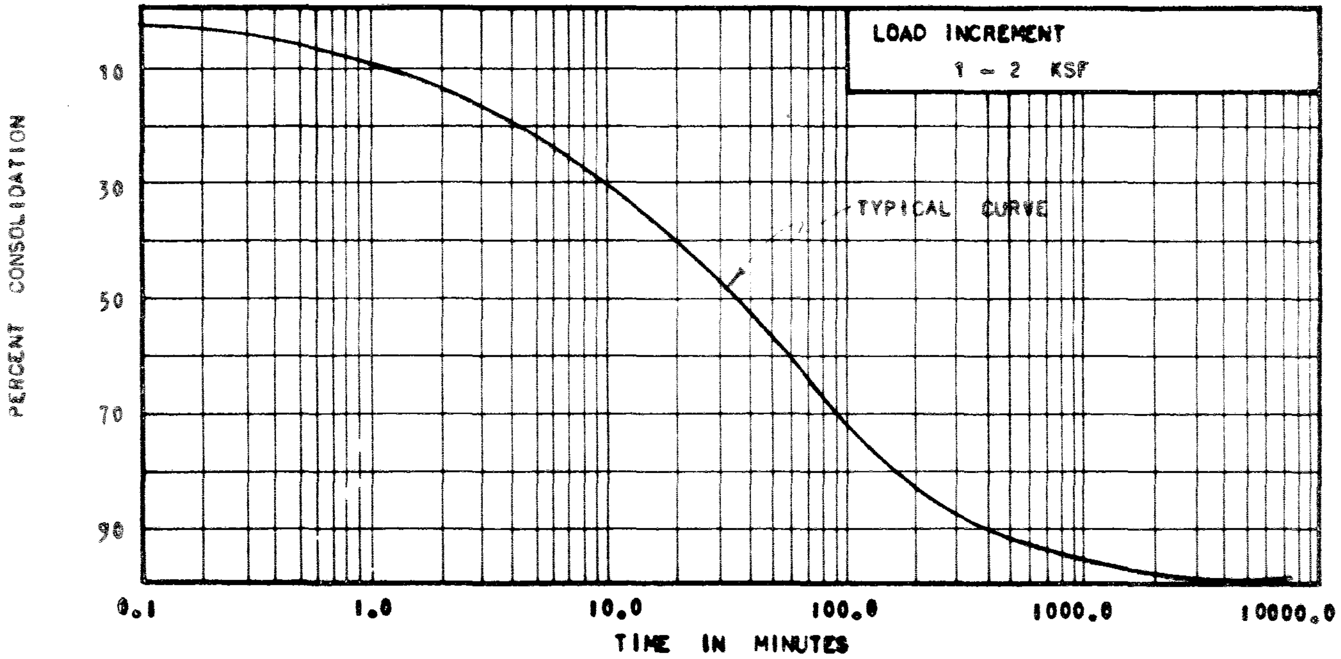


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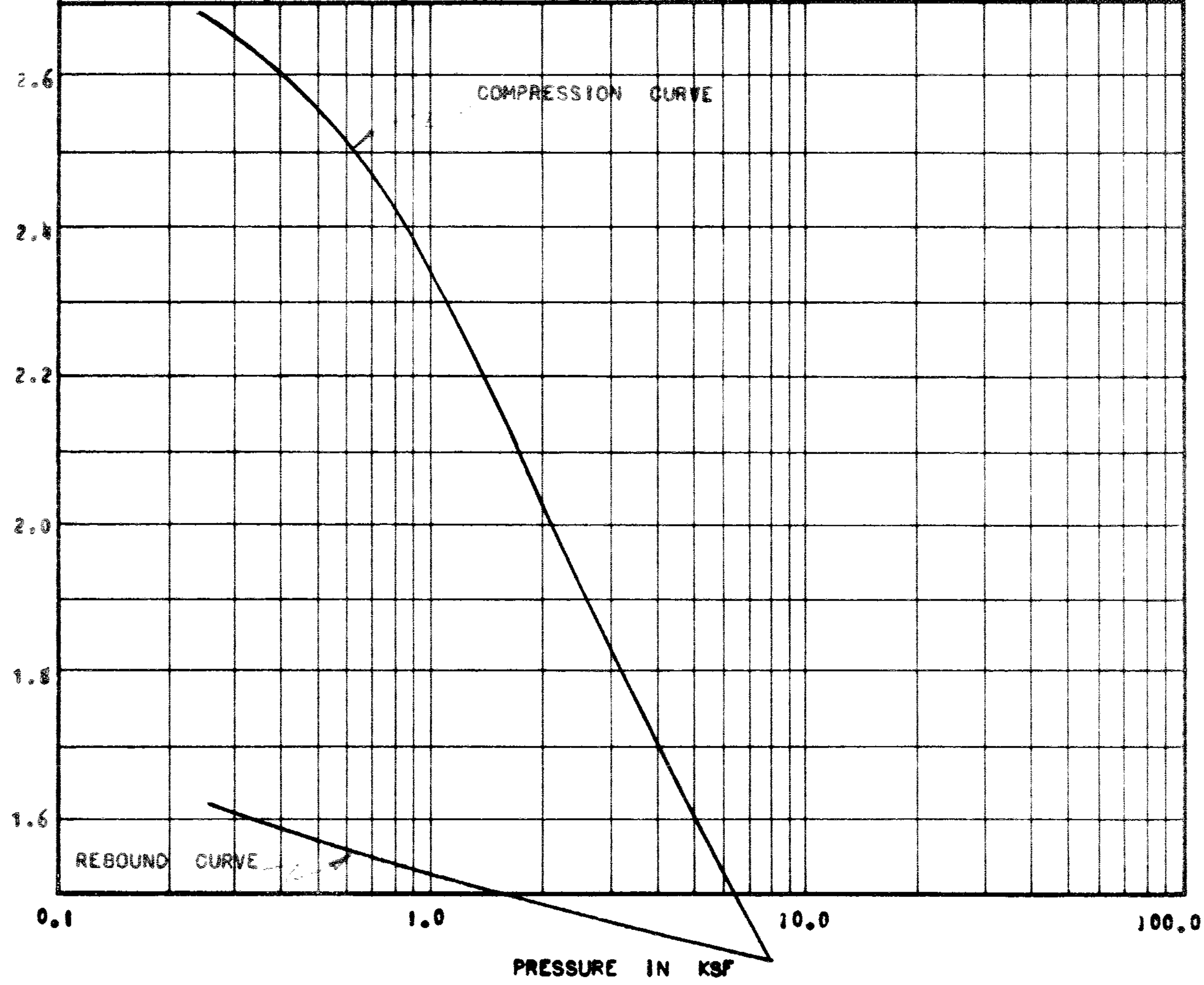
FIG. 73

SAMPLE NO. 44-3-4	SUMMARY OF TEST RESULTS					
	SPECIFIC GRAVITY	MOISTURE CONTENT, (%)	DRY DENSITY (PCF)	PER CENT OF SATURATION, (%)	HEIGHT (IN.)	DIAMETER (IN.)
INITIAL	ASSUMED 2.73	99.4	45.6	99.3	0.947	1.94
FINAL		62.9	63.1	100.0	0.685	

$e_0 = 2.73$



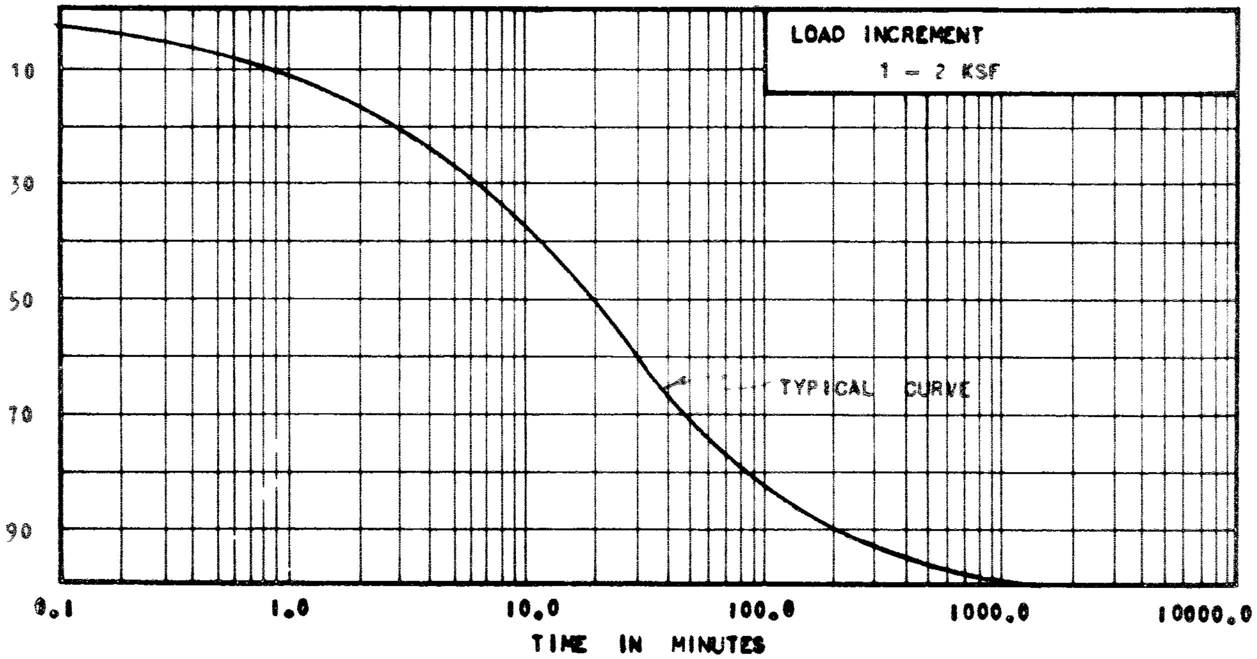


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FIG. 74

SAMPLE NO. B2-4-4	SUMMARY OF TEST RESULTS					
	SPECIFIC GRAVITY	MOISTURE CONTENT, (%)	DRY DENSITY (PCF)	PER CENT OF SATURATION, (%)	HEIGHT (IN.)	DIAMETER (IN.)
INITIAL	2.72	36.0	86.1	100.8	0.949	1.94
FINAL	(ASSUMED)	35.8	86.9	100.0	0.940	

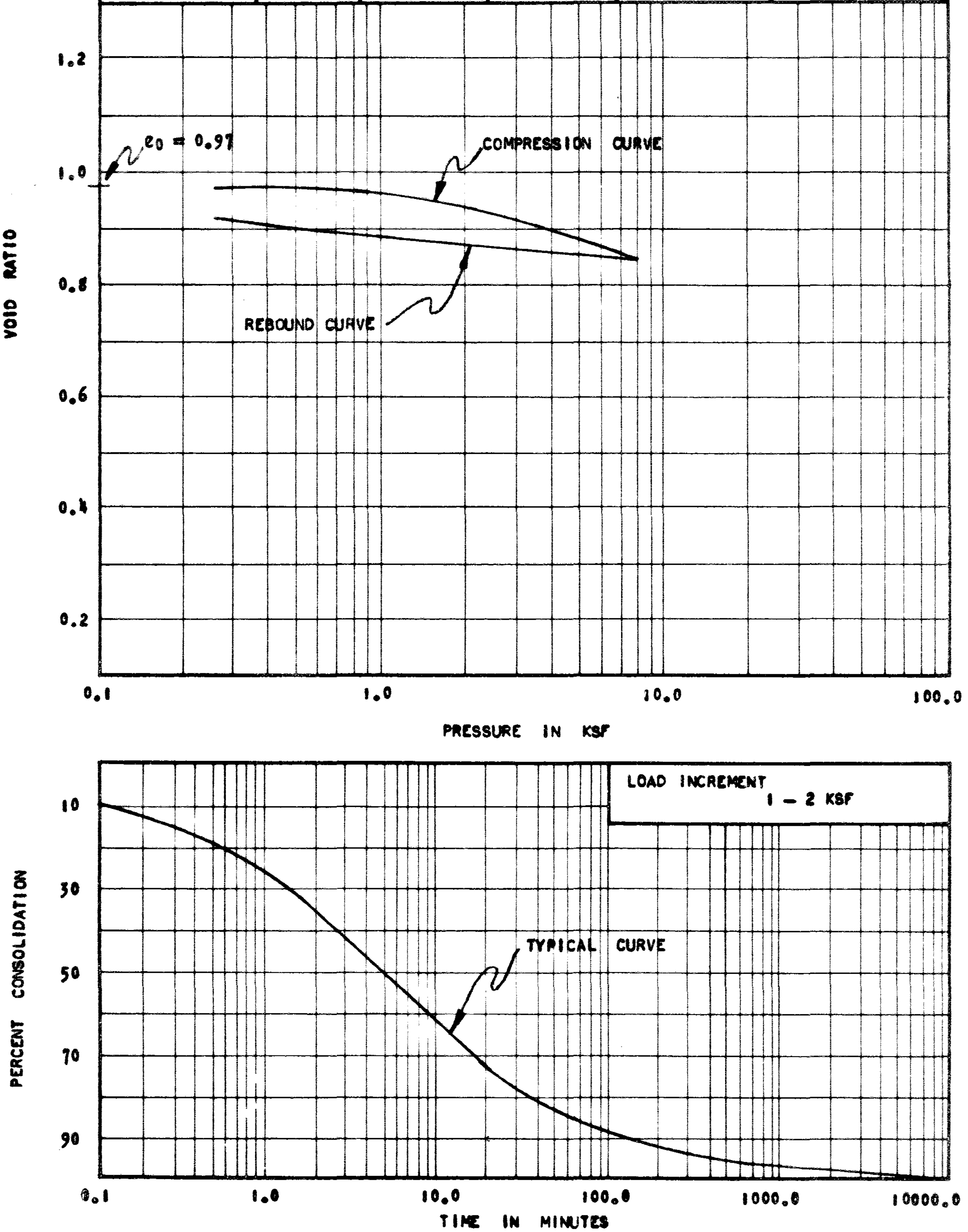

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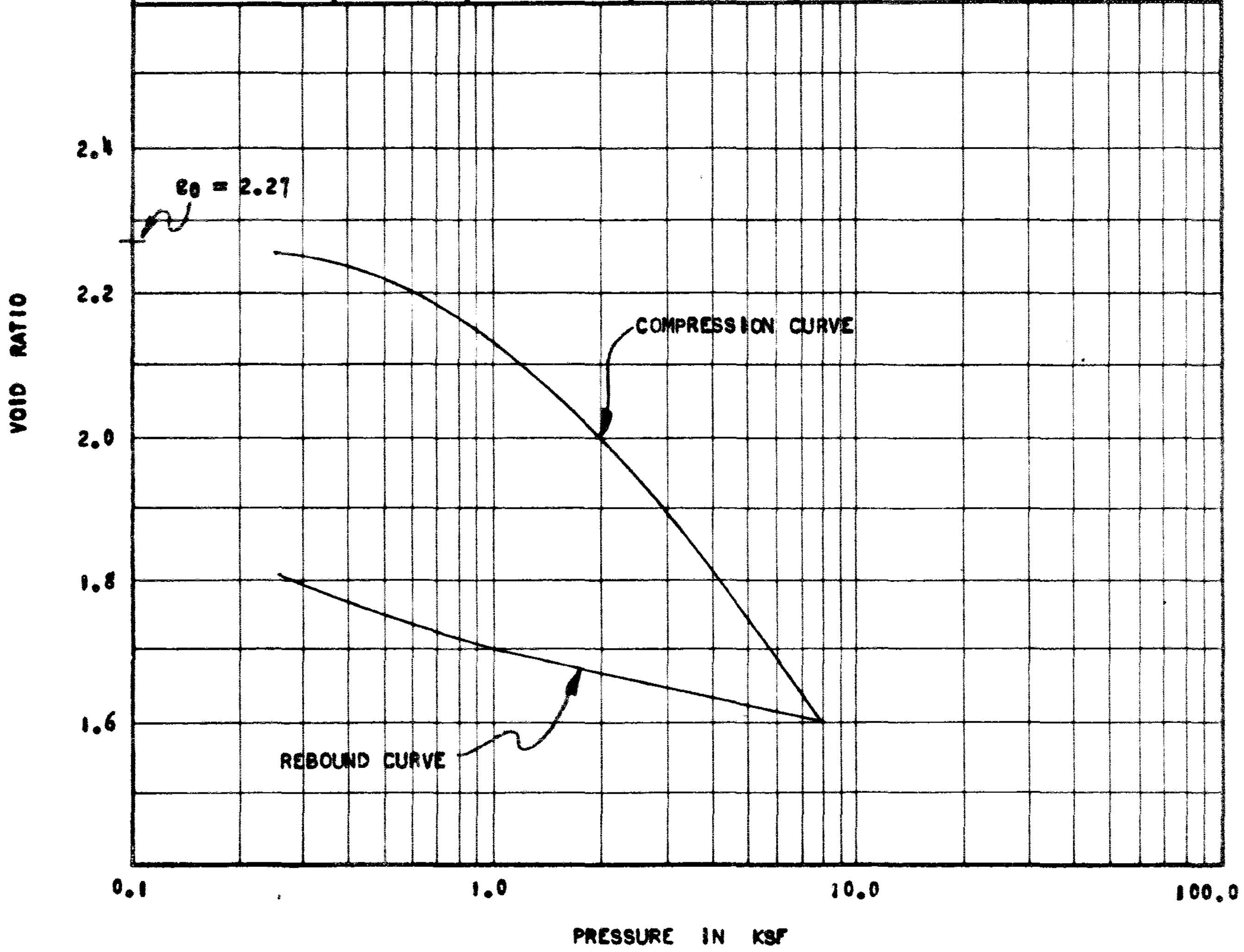
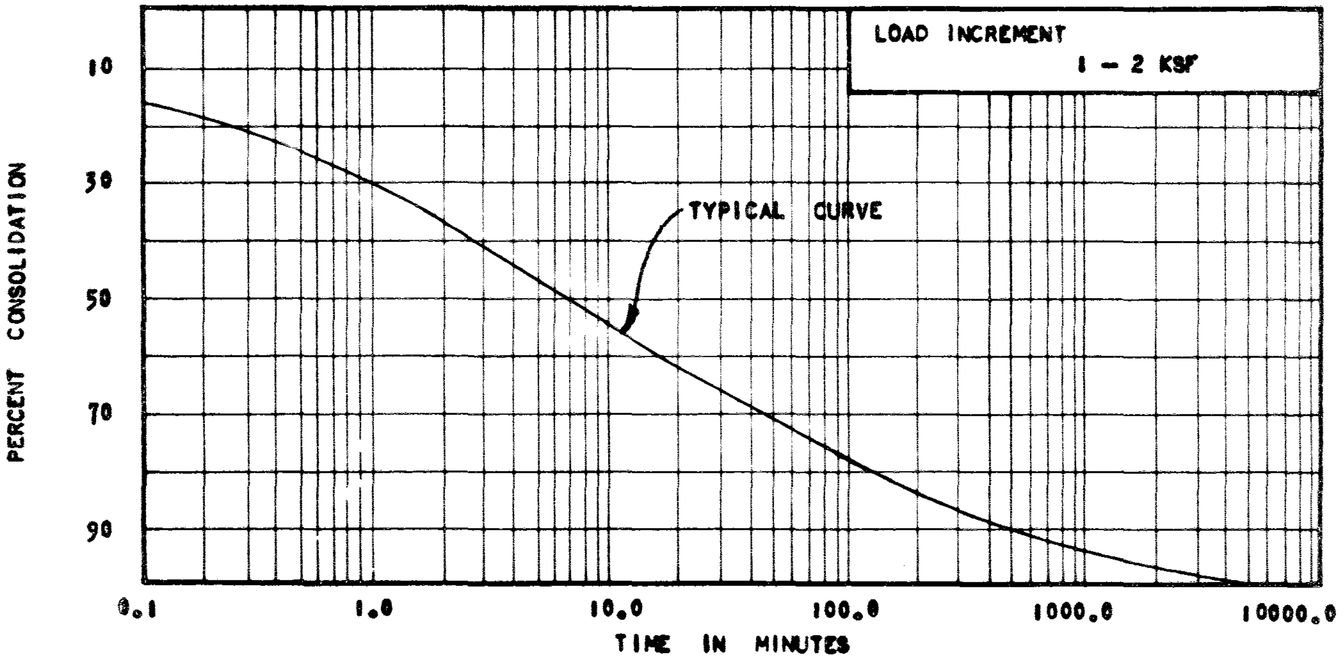
FIG. 9

SAMPLE NO.	SUMMARY OF TEST RESULTS					
B5-2-4	SPECIFIC GRAVITY	MOISTURE CONTENT, (%)	DRY DENSITY (PCF)	PER CENT OF SATURATION, (%)	HEIGHT (IN.)	DIAMETER (IN.)
INITIAL	2.55 (ASSUMED)	84.4	48.6	94.8	0.953	1.94
FINAL		71.3	55.7	97.9	0.833	





CONSOLIDATION TEST

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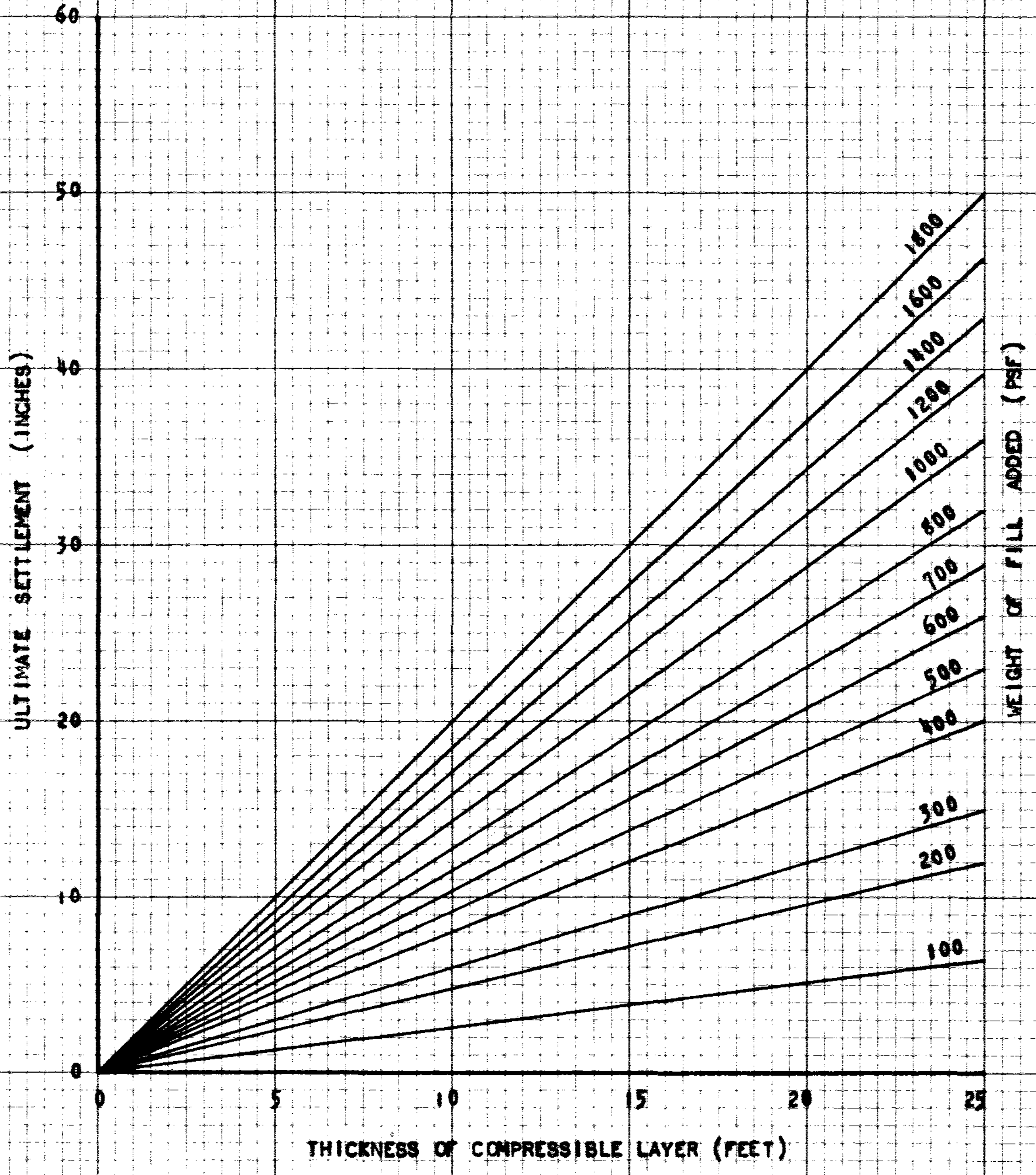
FIG. 76

K+E 10 X 10 TO THE INCH 359-5
KEUFFEL & ESSER CO. MADE IN U.S.A.



GUADALUPE RIVER DEVELOPMENT
REACH # 1

FIG. 77 - ULTIMATE SETTLEMENT VS THICKNESS OF COMPRESSIBLE LAYER FOR VARIOUS SURCHARGES

4246

✻ 4246

LOGARITHMIC 359-7
3 CYCLES X 70 DIVISIONS

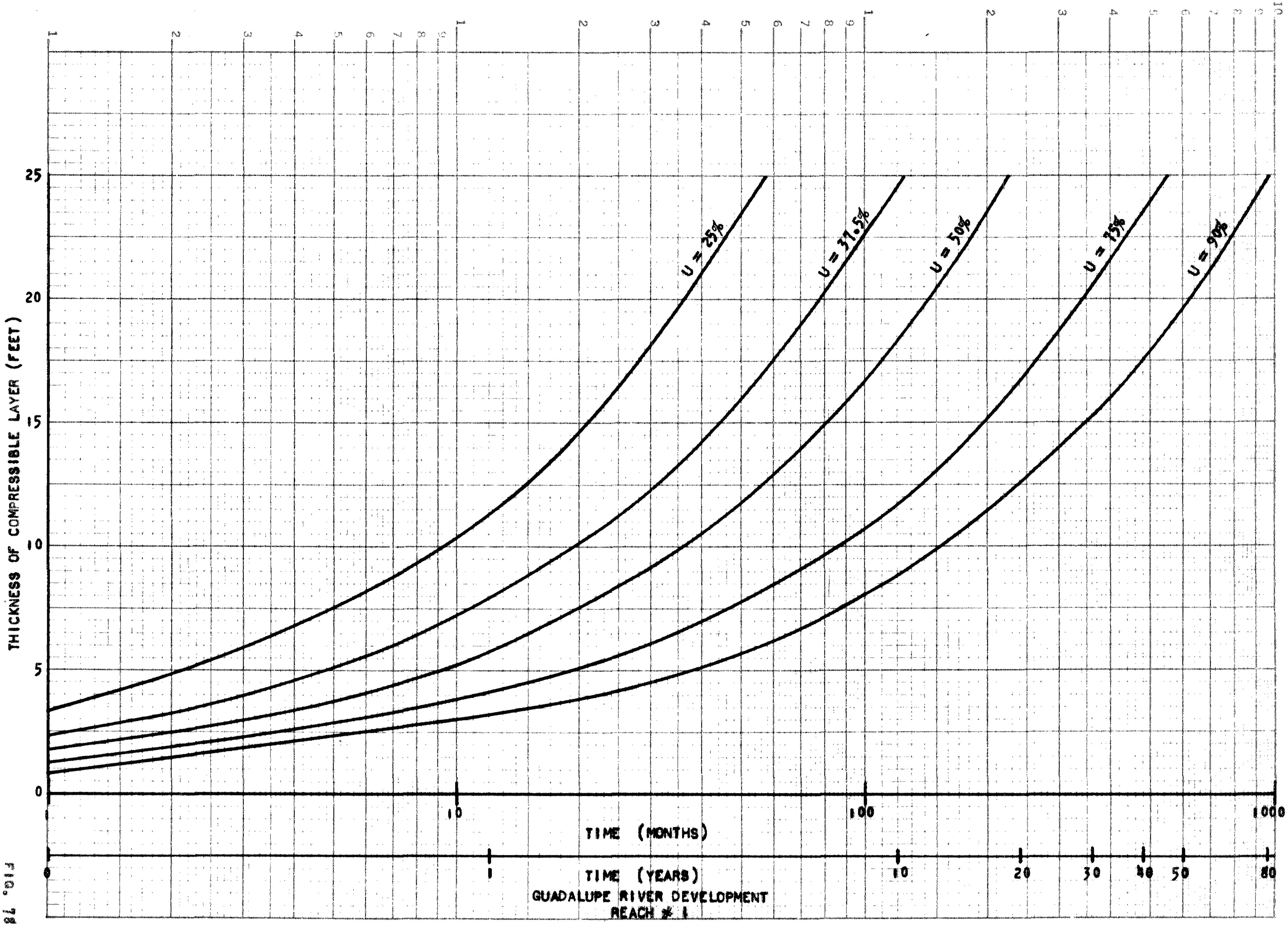
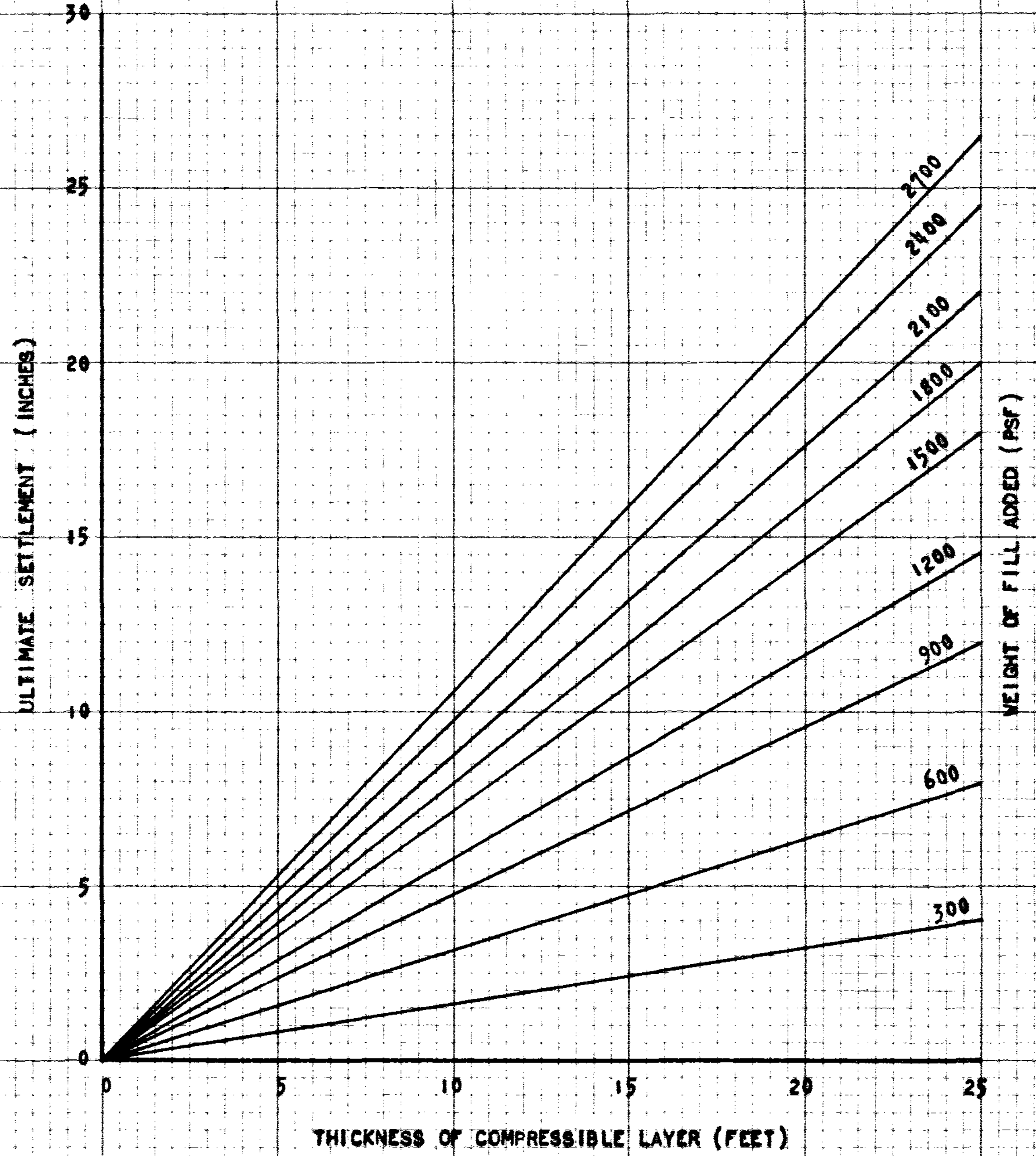


FIG. 78 — THICKNESS OF COMPRESSIBLE LAYER VS TIME FOR VARIOUS DEGREES OF CONSOLIDATION (SINGLE DRAINAGE)

K+E 10 X 10 TO THE INCH 359-5
KEUFFEL & ESSER CO. MADE IN U.S.A.



GUADALUPE RIVER DEVELOPMENT
GOLD STREET BRIDGE

FIG. 79 – ULTIMATE SETTLEMENT VS THICKNESS OF COMPRESSIBLE LAYER FOR VARIOUS SURCHARGES

4246

4246

SEMI-LOGARITHMIC 359-71
KEUFFEL & ESSER CO. MADE IN U.S.A.
3 CYCLES X 70 DIVISIONS

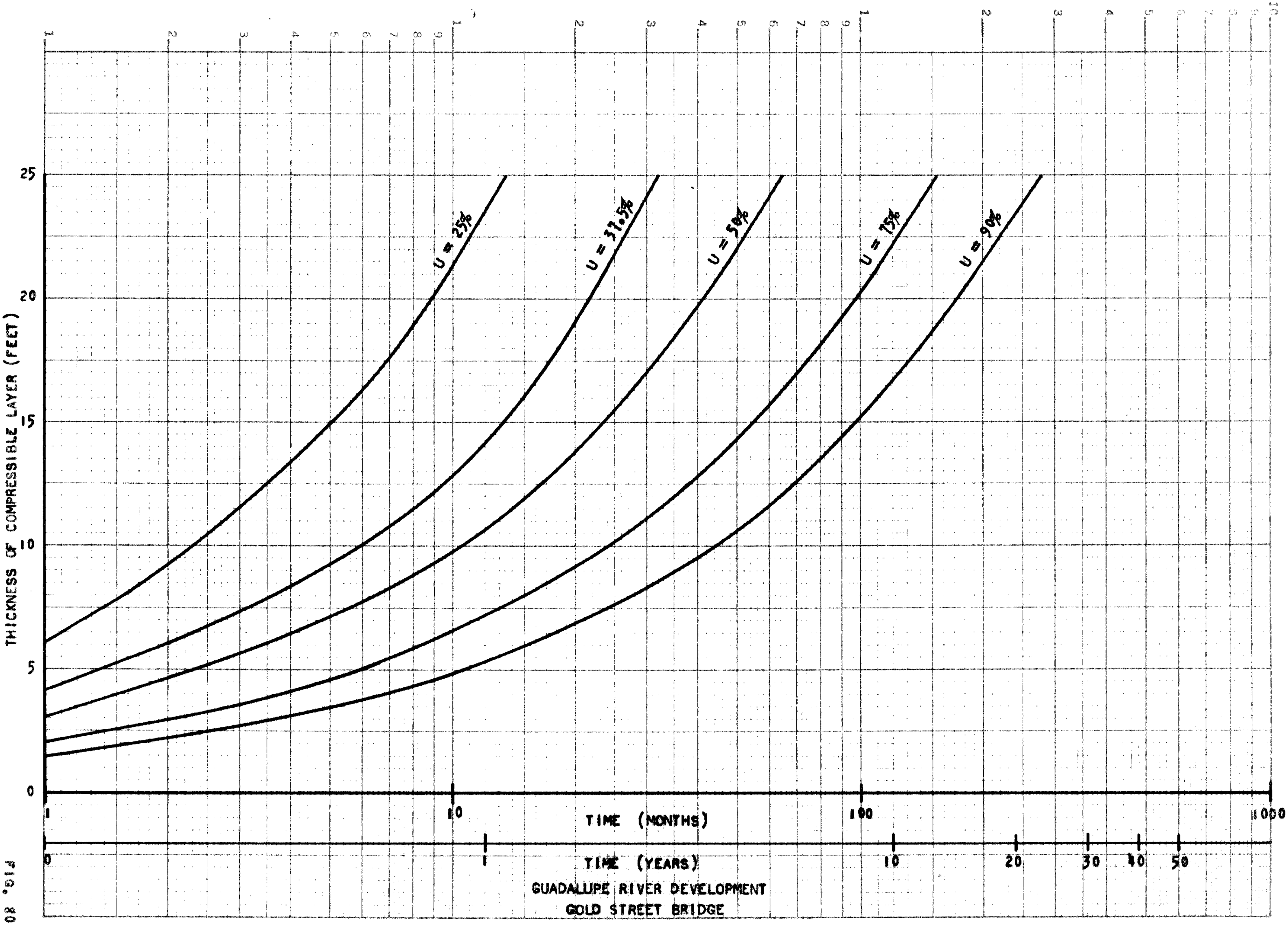


FIG. 80

FIG. 80 — THICKNESS OF COMPRESSIBLE LAYER VS TIME FOR VARIOUS DEGREES OF CONSOLIDATION (SINGLE DRAINAGE)

K+E 10 X 10 TO THE INCH 359-5
KEUFFEL & ESSER CO. MADE IN U.S.A.

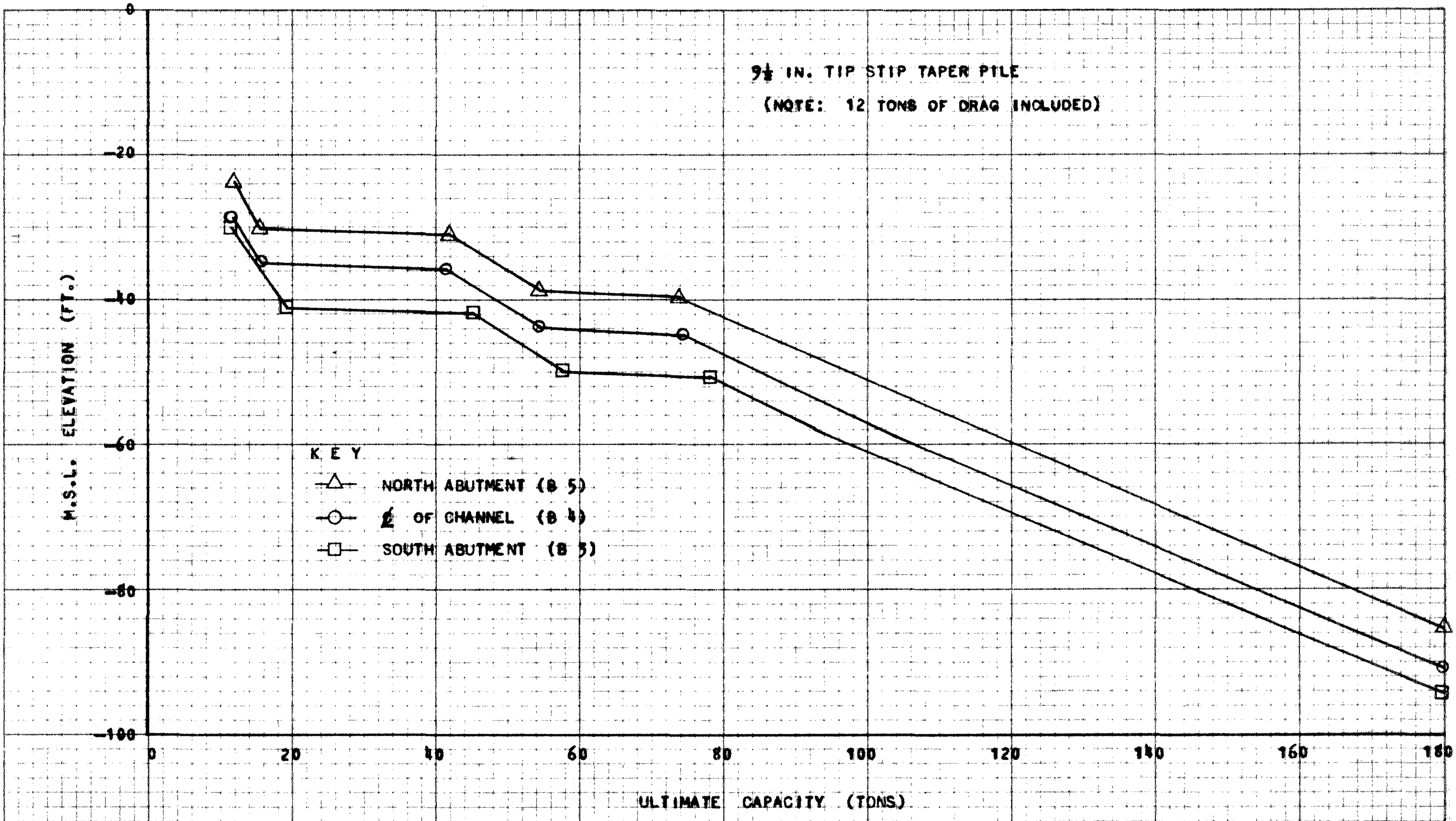
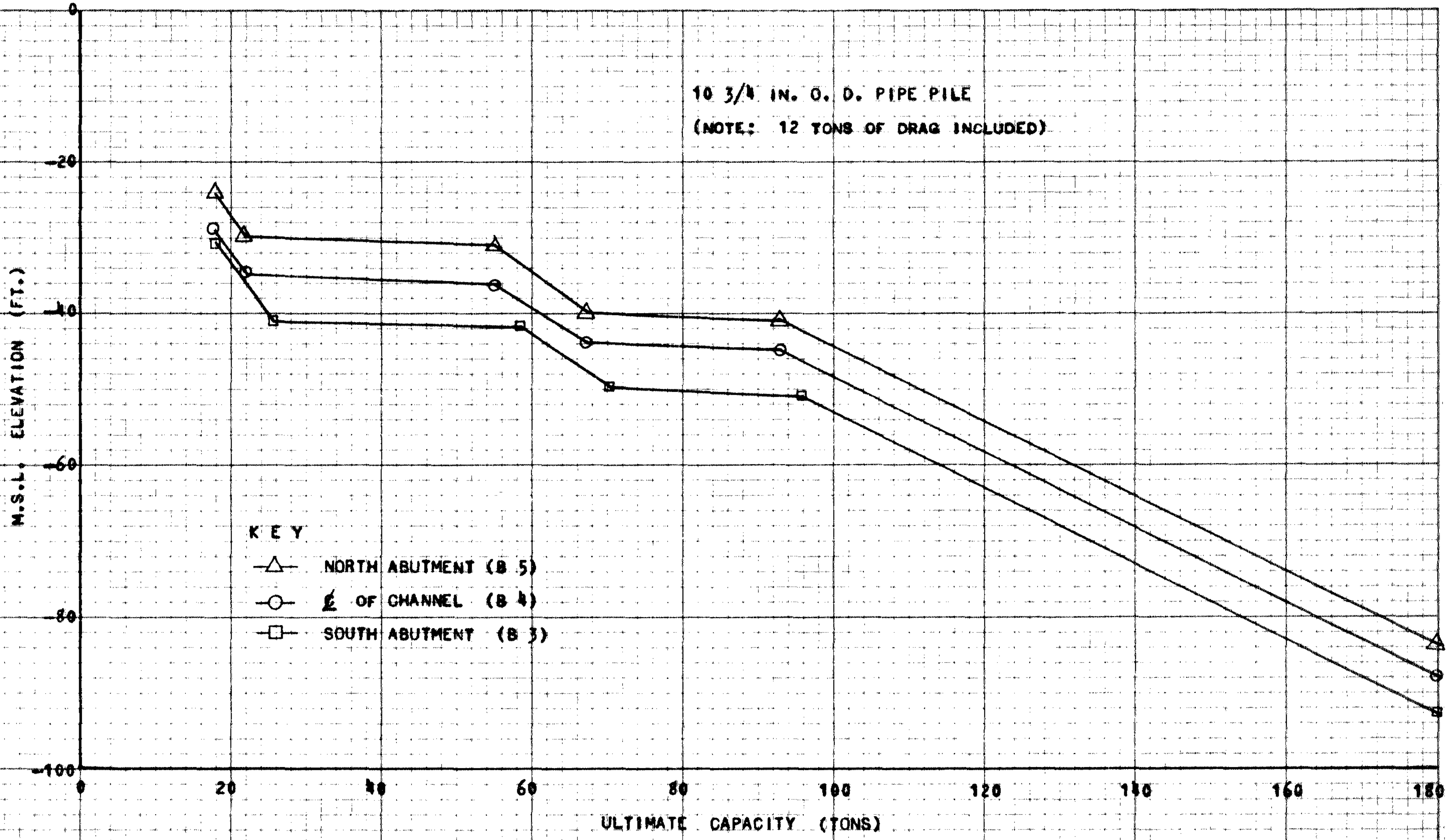


FIG. 81 – PILE CAPACITY VS DEPTH
GOLD STREET BRIDGE

FIG. 81

K+E 10 X 10 TO THE INCH 359-5
KEUFFEL & ESSER CO. MADE IN U.S.A.



10 3/4 IN. O. D. PIPE PILE
(NOTE: 12 TONS OF DRAG INCLUDED)

FIG. 82 – PILE CAPACITY VS DEPTH
GOLD STREET BRIDGE

FIG. 8

K+E 10 X 10 TO THE INCH 359-5
KEUFFEL & ESSER CO. MADE IN U.S.A.

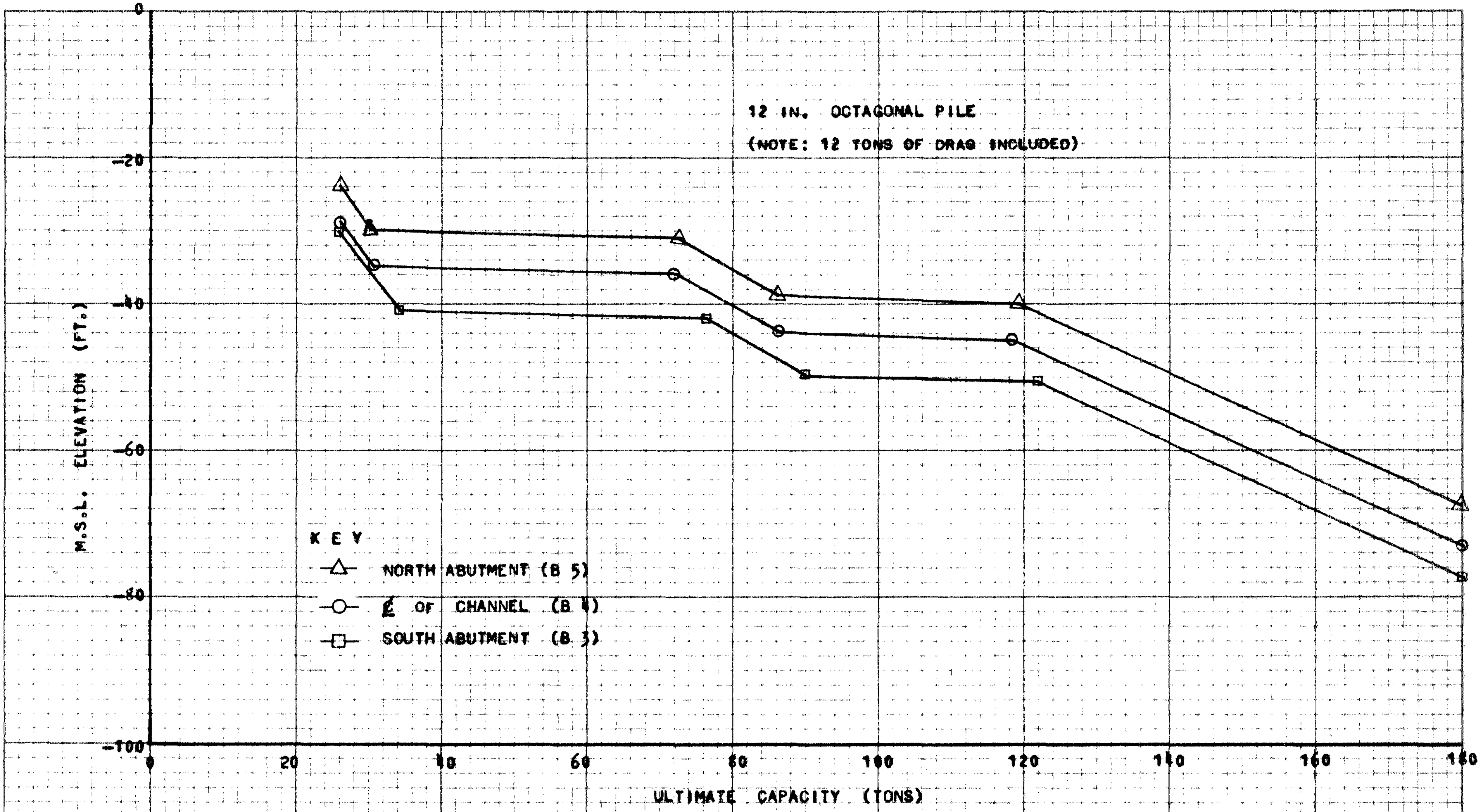


FIG. 8 — PILE CAPACITY VS DEPTH
GOLD STREET BRIDGE

FIG. 8